CHAPTER 7

CHANNELS

Chapter 7 Channels
Table Of Contents

Chapter 7 Channels
Table Of Contents

7.7 -- References

Appendix A

Appendix B

Appendix C

Appendix D

Appendix E

Appendix F

Appendix G

Appendix H

Appendix I

7.1 Introduction

7.1.1 Purpose

This chapter provides a discussion of the hydraulic behavior of channels. Guidance in the analysis and design of channels is provided through:

- presentation of the appropriate ADOT policy, philosophy and design criteria,
- discussion of geomorphic factors that affect channel behavior,
- presentation of an outlined design procedure,
- discussion of analysis of channels by computer programs such as HEC-RAS, and
- demonstration of design techniques by example problems.

7.1.2 Objectives in hydraulic design of channels.

An objective in the hydraulic design of channels is to provide a means to convey concentrated flows in a controlled manner with acceptable impacts to the environment. The objective in performing a hydraulic design or analysis of a channel is to assess:

- the hydraulic forces and predict the future behavior of streams and channels,
- impacts caused by changes in water surface profiles,
- impacts caused by changes in lateral flow distributions,
- impacts caused by changes in velocity or direction of flow,
- need for and value of channel linings for the control of erosion.

7.1.3 Concepts

Open channels are a natural or man-made conveyance for water in which:

- the water surface is exposed to the atmosphere, and
- the gravity force component in the direction of motion is the driving force.

There are various types of open channels encountered by the designer of transportation facilities:

- stream channel,
- roadside channel or ditch,
- irrigation channel,
- drainage ditch, and
- culvert in low flow.

7.1 Introduction (continued)

7.1.3 Concepts (continued)

Stream channels are:
- usually natural channels with their size and shape determined by natural forces,
- usually compound in cross section with a main channel for conveying low flows and a floodplain to transport flood flows, and
- usually shaped geomorphologically by the long-term history of sediment load and water discharge that they experience.

Artificial channels include roadside channels, irrigation channels and drainage ditches that are:
- man-made channels with regular geometric cross sections, and
- unlined, or lined with artificial or natural material to protect against erosion.

While the principles of open channel flow are the same regardless of the channel type, stream channels and artificial channels (primarily roadside channels) will be treated separately in this chapter as needed. The term roadside channel or ditch is used for those elements that collect and convey surface or sheet flow of storm water runoff from the highway and adjacent lands. Stream channel is used for elements that convey concentrated stream flows. Usually the alignment and profile of roadside channels and ditches is governed by the highway cross-section description. Relocated stream channels usually have an alignment independent of the roadway.

7.1.4 Design Goals and Considerations

7.1.4.1 ADOT Design Goals

Hydraulic design associated with natural channels and roadway ditches is a process that identifies and evaluates alternatives according to established criteria.

The range of design channel discharges shall be selected based on class of roadway, consequences of traffic interruption, flood hazard risks, economics, and local site conditions.

- Safety of the general public shall be an important consideration in the selection of cross-sectional geometry of artificial drainage channels.
- A stable channel is the goal for all man-made channels.
- Environmental impacts of channel modifications, including disturbance of habitat, wetlands, and streambank stability shall be assessed.
- The design of man-made drainage channels shall consider the frequency and type of maintenance expected and provide for access of maintenance equipment.
- Changes in water surfaces shall not significantly increase flood damage to property,
- Changes in velocity should not significantly alter the channel behavior nor significantly increase damage to adjacent property.

7.1 Introduction (continued)

7.1.4.2 Design Criteria

7.1.4.2.1 General

The following criteria apply to the design of hydraulic structures in natural or roadside channels and may be altered as appropriate when approved by the Drainage Section.

- As necessary, the hydraulic effects of flood plain encroachments should be evaluated for a peak discharge of the 100-year recurrence interval on any highway facility. Other discharges may need to be reviewed for a full understanding of the impact of the facility on the stream and surrounding environment.

- The effects of changes such as channel realignment, slope modification, section modification, and conveyance alteration must be adequately analyzed. Adverse effects such as bend and bank instability evaluated for possible mitigation requirements.

- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander pattern, roughness, sediment transport, and slope should, in so far as practicable, conform to the existing conditions. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

- Where overtopping would permit storm water to breakout of ADOT right-of-way, the minimum freeboard shall be 1-foot.

  For channels with flows having Froude Numbers equal to or greater than 0.86, the minimum freeboard should be the larger of 1-foot or the value

  $$F = 0.20*(y+v^2/2g)$$

  where:

  F = desired freeboard (1 ft minimum), ft;
  y = depth of flow, ft;
  v = mean velocity, ft/s;
  g = acceleration due to gravity, 32.2 ft/sec^2.

  For leveed channels where the water surface elevation is higher than natural ground, provide an additional 1-foot of freeboard to accommodate surface irregularities and alignment adjustments.

7.1 Introduction (continued)

7.1.4.2 Design Criteria (continued)

- In earthen channels, side slopes shall be flatter than the angle of repose of the soil and/or lining and shall be 2:1 or flatter in the case of rock-riprap lining.

- Streambank stabilization shall be provided, when appropriate, as a result of any stream disturbance such as encroachment and shall extend to include both upstream and downstream banks as well as the local site.

- Flexible linings shall be designed according to the method of allowable tractive force.

- The design discharge for channel linings shall be based on the hazard associated with the failure of lining. For lining that protects the highway embankment from erosion, the design frequency shall be not less that the operational frequency of the highway. Where the failure of roadside channels would increase the flood hazard of adjacent properties the channel shall safely convey the 100-year event within the ADOT r/w. The minimum design discharge for permanent roadside ditch linings shall be a 10-year frequency while temporary linings may be a 2-year frequency flow.

7.2 Stream Morphology

7.2.1 Introduction

The form assumed by a natural stream, which includes its cross-sectional shape as well as its planform, is a function of many variables for which cause-and-effect relationships are difficult to establish. The natural stream channel will assume a geomorphological form that will be compatible with the sediment load and discharge history that it has experienced over time. The stream may be graded or in equilibrium with respect to long time periods, which means that on the average it discharges the same amount of sediment that it receives although there may be short-term adjustments in its bedforms in response to flood flows. On the other hand, the stream reach of interest may be aggrading or degrading as a result of deposition or scour in the reach, respectively. The planform of the stream may be straight, braided, or meandering.

To the extent that a highway structure disturbs this delicate balance by encroaching on the natural channel, the consequences of flooding, erosion, and deposition can be significant and widespread. The hydraulic analysis of a proposed highway structure should include a consideration of the extent of these consequences.

These complexities of stream morphology can be assessed by inspecting aerial photographs and topographic maps for changes in slope, width, depth, meander form, and bank erosion with time. A qualitative assessment of the river response to proposed changes is possible through a thorough knowledge of river mechanics and accumulation of engineering experience. Equilibrium sediment load calculations can be made by a variety of techniques and compared from reach to reach to detect an imbalance in sediment inflow and outflow and thus identify an aggradation/degradation problem. References (FHWA, 1990) should be consulted to evaluate the problem and propose mitigation measures. The proposed methodology should be approved before beginning any in-depth study of a site.

7.2 Stream Morphology (continued)

7.2.2 Levels Of Assessment

The analysis and design of a stream channel will usually require an assessment of the existing channel and the potential for problems as a result of the proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream. Observation of the existing stream is the best means of identifying potential locations for channel bank erosion and subsequent channel stabilization. Analytical methods for the evaluation of channel stability can be classified as either hydraulic or geomorphic, and it is important to recognize that these analytical tools should only be used to substantiate the erosion potential indicated through observation. Brief descriptions of the three levels of assessment are as follows:

Level 1
Qualitative assessment involving the application of geomorphic concepts to identify potential problems and alternative solutions. Data needed may include historic information, current site conditions, aerial photographs, old maps and survey notes, bridge design files, maintenance records, and interviews with long-time residents.

Level 2
Quantitative analysis combined with a more detailed qualitative assessment of geomorphic factors generally includes water surface profile and scour calculations. This level of analysis will be adequate for most locations if the problems are resolved and relationships between different factors affecting stability are adequately explained. Data needs will include Level 1 data in addition to the information needed to establish the hydrology and hydraulics of the stream.

Level 3
Complex quantitative analysis based on detailed mathematical modeling and possibly physical hydraulic modeling. These methods are necessary only for high-risk locations, extraordinarily complex problems, and possibly after the fact analysis where losses and liability costs are high. This level of analysis may require professionals experienced with mathematical modeling techniques for sediment routing and/or physical modeling. Data needed will require Level 1 and 2 data as well as field data on bed load and suspended load transport rates and properties of bed and bank materials such as size, shape, gradation, fall velocity, cohesion, density, and angle of repose.

7.2.3 Factors That Affect Stream Stability

An alluvial stream is continually changing its position and slope as a consequence of hydraulic factors acting on its bed and banks. These changes may be slow or rapid and may result from natural environmental changes or man's activity.

7.2 Stream Morphology (continued)

7.2.3 Factors That Affect Stream Stability (continued)

A study of the plan and profile is useful in understanding stream morphology. Factors that affect the stream shape and stability and, potentially, bridge and highway stability at stream crossings, can be classified as geomorphic factors and hydraulic factors.

Geomorphic Factors:

- Size
- Flow habit
- Bed material
- Valley setting
- Flood plains
- Natural levees
- Apparent incision
- Channel boundaries
- Cut banks
- Tree cover on banks
- Sinuosity
- Degree of braiding
- Degree of anabranching
- Width variability
- Bar development

Figure 7-1 depicts examples of the various geomorphic factors.

Hydraulic Factors.

- Magnitude, frequency and duration of floods.
- Bed configuration: ripples, dunes, plane bed, antidunes
- Resistance to flow: Manning' n
- Water surface profiles.

Figure 7-2 depicts the changes in channel classification and relative stability as related to hydraulic factors.

Rapid and unexpected changes may occur in streams in response to man's activities in the watershed. Changes in perviousness can alter the hydrology of a stream, sediment yield, and channel geometry. Channelization, stream channel straightening, stream levees and dikes, bridges and culverts, reservoirs, and changes in land use can have major effects on stream flow, sediment transport, and channel geometry and location. Knowing that man's activities can influence stream stability can help the designer anticipate some of the problems that can occur. Natural disturbances such as floods, drought, earthquakes, landslides, volcanoes, and forest fires can also cause large changes in sediment load and thus major changes in the stream channel. Although difficult to plan for such disturbances, it is important to recognize that when natural disturbances do occur, it is likely that changes will also occur to the stream channel.

7.2 Stream Morphology (continued)

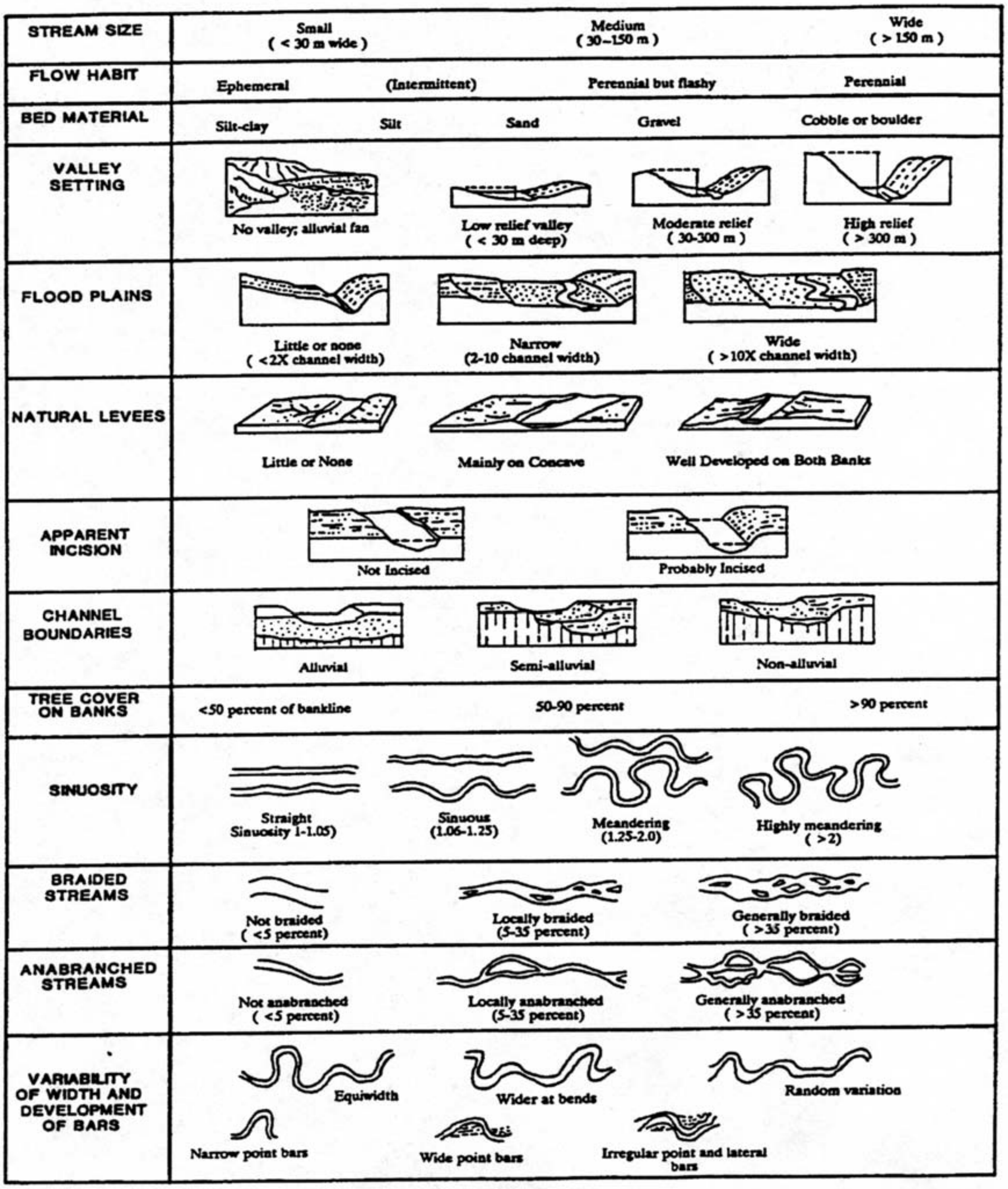


Figure 7-1 Geomorphic Factors That Affect Stream Stability

Source: Adapted From Brice and Blodgett, 1978

7.2 Stream Morphology (continued)

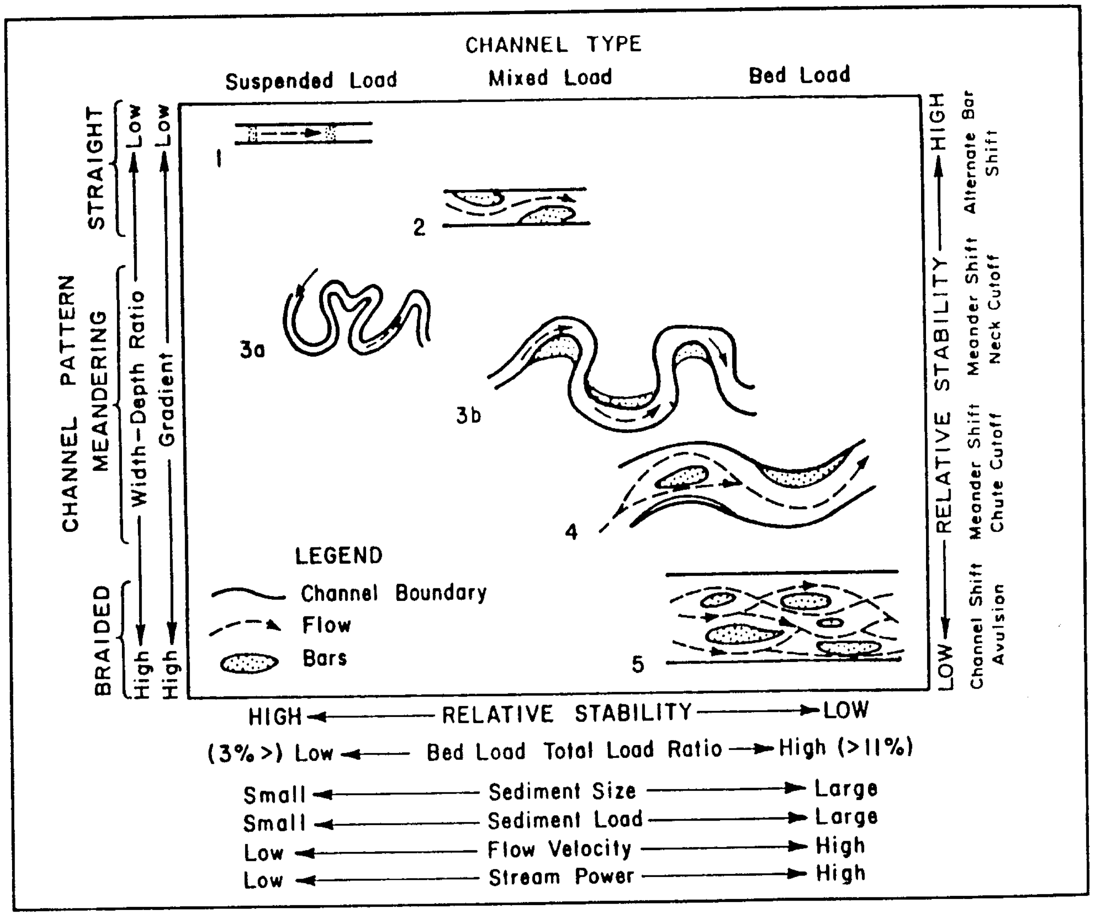


Figure 7-2 Channel Classification And Relative Stability As Hydraulic Factors Are Varied
Source: Atter, Shen et al., 1981

7.2 Stream Morphology (continued)

7.2.4 Stream Classification

David L. Rosgen (Rosgen, 1994 and 1996) developed a stream classification system that is a widely accepted method for classifying streams. This system may be of use in determining the sensitivity of a site to lateral migration, and change rates. This information may be used to address the scour susceptibility of foundation elements located outside the floodplain. Aa, A, and B-type streams will not normally be very active in lateral migration. Foundation elements that are outside the floodplain may be able to be constructed at elevations shallower than the thalweg of the stream. This will require approval of the Drainage Section. D, F, and G-type streams will usually be very active in lateral migration and bank instability: they will need to have the foundation elements be designed with the possibility of being exposed to the main-channel flow at some time in the future.

Rosgen's stream classification system is delineated initially into major, broad, stream categories of A – G as shown in Table 7-1. At this level, which Rosgen refers to as level I, the classification system uses the entrenchment ratio, sinuosity, width/depth ratio, and the channel slope as the delineative criteria for classifying a river. The entrenchment ratio is the ratio of the width of the flood-prone area to the bankfull surface width of the channel. The flood-prone area is defined as the width measured at an elevation that is determined at twice the maximum bankfull depth. The width/depth ratio is the ratio of bankfull channel width to bankfull mean depth. The bankfull mean depth is the bankfull area divided by the bankfull channel width. Sinuosity is the ratio of stream length to valley length and it can also be described as the ratio of valley slope to channel slope. Slope is the water surface slope and can be determined by measuring the difference in water surface elevation per unit stream length. At the broad level classification the slope can be estimated from USGS quadrangle maps.

The broad level classifications are then broken into sub-classes based on the dominant bed material. The stream types are assigned numbers related to the size of the dominant bed material such that 1 is bedrock, 2 is boulder, 3 is cobble, 4 is gravel, 5 is sand, and 6 is silt/clay. This produces 41 major stream types as shown in Table 7-2. Rosgen's classification system also incorporates a continuum concept. The continuum concept is applied where delineative criteria values outside the normal range are encountered but do not warrant a unique stream type. This yields the following sub-categories based on slope: a+ (steeper than 0.10), a (0.04 – 0.099), b (0.02 – 0.039), c (flatter than 0.02), and c- (flatter than 0.001). The continuum concept also allows the entrenchment ratio and sinuosity to vary by $\pm$0.2 unit and sinuosity can vary by $\pm$2.0 units. The expanded classification system that incorporates the continuum concept is shown in Table 7-3. Rosgen refers to the classifications shown in Tables 7-2 and 7-3 as level III.

Stream Type	General Description	Entrenchment Ratio	W/D Ratio	Sinuosity	Slope	Landform/Soils/Features
Aa+	Very steep, deeply entrenched, debris transport streams.	<1.4	<12	1.0 - 1.1	>0.10	Very high relief. Erosional, bedrock or depositional features; debris flow potential. Deeply entrenched streams. Vertical steps with deep scour pools; waterfalls.
A	Steep, entrenched, cascading, step/pool streams. High energy/debris transport associated with depositional soils. Very stable if bedrock or boulder dominated channel	<1.4	<12	1.0 -1.2	0.04 - 0.10	High relief. Erosional or depositional and bedrock forms. Entrenched and confined streams with cascading reaches. Frequently spaced, deep pools in associated step/pool bed morphology.
B	Moderately entrenched, moderate gradient, riffle dominated channel with infrequently spaced pools. Very stable plan and profile. Stable banks.	1.4 - 2.2	>12	>1.2	0.02 - 0.039	Moderate relief, colluvial deposition and/or residual soils. Moderate entrenchment and width/depth ratio. Narrow, gently sloping valleys. Rapids predominate with occasional pools.
C	Low gradient, meandering, point-bar, riffle/pool, alluvial channels with broad well defined floodplains	>2.2	>12	>1.4	<0.02	Broad valleys with terraces in association with floodplains, alluvial soils. Slightly entrenched with well-defined meandering channels. Riffle/pool bed morphology.
D	Braided channel with longitudinal and transverse bars. Very wide channel with eroding banks.	n/a	>40	N/a	<0.04	Broad valleys with alluvial and colluvial fans. Glacial debris and depositional features. Active lateral adjustment with abundance of sediment supply.
DA	Anastomosing (multiple channels) narrow and deep with expansive well vegetated floodplain and associated wetlands. Very gentle relief with highly variable sinuosities. Stable streambanks.	>2.2	Highly variable	Highly variable	<0.005	Broad, low gradient valleys with fine alluvium and/or lacustrine soils. Anastomosed geologic control creating fine deposition with well-vegetated bars that are laterally stable with broad wetland floodplains.
E	Low gradient, meandering riffle/pool stream with low width/depth ratio and little deposition. Very efficient and stable. High meander width ratios.	>2.2	<12	>1.5	<0.02	Broad valley/meadows. Alluvial materials with floodplain. Highly sinuous with stable, well vegetated banks. Riffle/pool morphology with very low width/depth ratio.
F	Entrenched meandering riffle/pool channel on low gradients with high width/depth ratio.	<1.4	>12	>1.4	<0.02	Entrenched in highly weathered material. Gentle gradients with a high width/depth ratio. Meandering, laterally unstable with high bank-erosion rates. Riffle/pool morphology.
G	Entrenched "gully" step/pool and low width/depth ratio on moderate gradients.	<1.4	<12	>1.2	0.02 – 0.039	Gully, step/pool morphology with moderate slopes and low width/depth ratio. Narrow valleys or deeply incised in alluvial or colluvial materials; i.e., fans or deltas. Unstable with grade control problems and high bank erosion rates.

Table 7-1 Summary of Delineative Criteria for Broad-Level Classification

Rosgen's River Classification										
Bed Material	Bedrock	A1a+	A1	B1	C1				F1	G1
	Boulder	A2a+	A2	B2	C2				F2	G2
	Cobble	A3a+	A3	B3	C3	D3		E3	F3	G3
	Gravel	A4a+	A4	B4	C4	D4	DA4	E4	F4	G4
	Sand	A5a+	A5	B5	C5	D5	DA5	E5	F5	G5
	Silt/Clay	A6a+	A6	B6	C6	D6	DA6	E6	F6	G6
Criteria	Entrenchment	<1.4	<1.4	1.4 - 2.2	>2.2	N/A	>4.0	>2.2	<1.4	<1.4
	Sinuosity	1.0 -1.1	1.0 - 1.2	>1.2	>1.2	N/A	Variable	>1.5	>1.2	>1.2
	Width/Depth	<12	<12	>12	>12	>40	<40	<12	>12	<12
	Slope	>0.10	0.04 - 0.099	0.02 - 0.039	<0.02	<0.04	<0.005	<0.02	<0.02	0.02 - 0.039

Table 7-2 Rosgen's River Classification System

		Single Thread Channels														Multiple Channels			
Entrenchment Ratio[1]		Entrenched (<1.4)						Moderately Entrenched (1.4 – 2.2)			Slightly Entrenched (>2.2)					N/A			
Width/Depth Ratio[2]		Low (<12)				Moderate -High (>12)		Moderate (>12)			Very Low (<12)		Moderate – High (>12)			Very High (>40)			Low (<40)
Sinuosity[1]		Low (<1.2)		Moderate (>1.2)		High (>1.2)		Moderate (>1.2)			Very High (>1.5)		High (>1.2)			Low (<1.2)			Low - High (1.2-1.5)
Broad Class		A		G		F		B			E		C			D			DA
Slope Range		>0.10	0.04 to 0.099	0.02 to 0.039	<0.02	0.02 to 0.039	<0.02	0.04 to 0.099	0.02 to 0.039	<0.02	0.02 to 0.039	<0.02	0.02 to 0.039	0.001 to 0.02	less than 0.001	0.02 to 0.039	0.001 to 0.02	less than 0.001	less than 0.005
CHANNEL MATERIAL	Bedrock	A1a+	A1	G1	G1c	F1b	F1	B1a	B1	B1c			C1b	C1	C1c-				
	Boulders	A2a+	A2	G2	G2c	F2b	F2	B2a	B2	B2c			C2b	C2	C2c-				
	Cobbles	A3a+	A3	G3	G3c	F3b	F3	B3a	B3	B3c	E3b	E3	C3b	C3	C3c-	D3b	D3		
	Gravel	A4a+	A4	G4	G4c	F4b	F4	B4a	B4	B4c	E4b	E4	C4b	C4	C4c-	D4b	D4	D4c-	DA4
	Sand	A5a+	A5	G5	G5c	F5b	F5	B5a	B5	B5c	E5b	E5	C5b	C5	C5c-	D5b	D5	D5c-	DA5
	Silt/Clay	A6a+	A6	G6	G6c	F6b	F6	B6a	B6	B6c	E6b	E6	C6b	C6	C6c-	D6b	D6	D6c-	DA6

[1]Values can vary by ± 0.2 unit [2]Values can vary by ± 2.0 unit

Table 7-3 Rosgen's Stream Classification System

7.3 Alluvial Streams

7.3.1 Classification

The purpose of the stream classification system is to assist the users in assessing stream stability and in choosing the appropriate sediment transport equation. The methods utilized are predicated on bed material sediment size and stream channel slope. Stream morphology and related channel patterns are directly influenced by the width, depth, velocity, discharge, slope, and roughness of channel material, sediment load and sediment size. Changes in any of these variables can result in altered channel patterns. As stream morphology is a result of these mutually adjustable variables, those most directly measurable were incorporated into Rosgen's criteria for stream classification. Stream channel patterns are classified based upon bed material size, channel gradients, and channel entrenchment and confinement.

7.3.1.1 Stream types

There are many properties by which streams may be classified into categories. 14 categories have been identified, see page 7-9, these categories are combined in groupings that represent a characteristic association of properties. A reason for using these classifications is to facilitate the assessment of the stream for engineering purposes, with particular regard to lateral stability.

The benefit is based on two premises: (1.) that lateral stability, as well as several other aspects of stream behavior is reflected in the physical appearance of the stream and its channel, and (2.) the best guide to the behavior of a stream is its behavior during the immediate past.

NOTES ON STREAM TYPES:
Five alluvial stream types have been identified.

These are TYPE A: Equiwidth, point-bar system
TYPE B: Wide bend, point-bar system
TYPE C: Braided, Point-bar system
TYPE D: Braided stream, without point bars.
TYPE E: Anabranched streams

Characteristics and engineering significance is presented in the next paragraphs.

7.3 Alluvial Streams (continued)

7.3.1 Classification (continued)

TYPE A: Equiwidth, point-bar system

The main characteristic is that it has point bars, with lateral bars being rare. It is not braided, and it may have any degree of sinuosity.

Engineering significance--It is the most stable of all stream types, meanders may gradually migrate. Rate of bed load transport is probably small in relation to suspended load.

TYPE B: Wide bend, point-bar system

In addition to many point bars, it may have a few lateral bars and be locally braided. Markings of point bars, if visible, will tend to be concentric. The stream may have any degree of sinuosity. Meanders may be of the neck or chute type.

Engineering significance--this type of stream may have straight reaches that are stable for decades; however, there is a potential for high rates of lateral migration at bends. There is substantial transport of bed material, either sand or gravel.

TYPE C: Braided, Point-bar system

This stream will have a mix of bars; point bars, lateral bars and mid-channel bars. The bars may be of sand, gravel or cobbles. It is locally or generally braided, but has a continuous thalweg. The thalweg may be either sinuous or meandering, may be fairly stable or shift dramatically during floods. The making of point bars tends to be irregular and not concentric. There may be variability in stream width. The main channel may be sinuous, but less than the thalweg.

Engineering significance--This type of stream has the potential for a very high rate of lateral erosion. Rapid movement of the thalweg or chute cutoffs of bends may occur and result in alignment problems. Potentially deep scour may occur at the thalweg, particularly if the bed is silt or sand. The transport of bed load (sand, gravel, or cobbles) probably exceeds the transport of suspended load.

TYPE D: Braided stream, without point bars.

This type of stream has no point bars at banks in the main channel. It will have many mid-channel and lateral bars, the flow may be completely divided. There will be scattered small islands, or islands more numerous than bars. There is usually random variation in the stream width.

Engineering significance--The channel tends to be wide and shallow, requires a long bridge confined by suitable countermeasures. The lateral erosion rates are low to moderate, but the point of erosion is not predictable. The braids shift at each high flow, and unexpected depths of scour may occur where braids join to form a deep channel. The load is transported mainly as bed load, either sand, gravel, or cobbles.

7.3 Alluvial Streams (continued)

7.3.1 Classification (continued)

TYPE E: Anabranched streams

The flow is distinctly divided into channels separated by large islands, which are usually covered with permanent vegetation. Anabranches are likely to be locally braided. Point bars are likely at bends in anabranches. The stream, as well as individual anabranches, may be straight, sinuous, or meandering.

Engineering significance--A long bridge is required unless the stream is crossed at a local point where it is not anabranched. If multiple structures are used, the percent of total flow at each bridge may not be predictable. The stability of each anabranch on streams differs greatly, and should be assessed as though an anabranch were an individual stream.

7.3.1.2 Stream Alignment

Streams may be further classed by considering their alignment. Whether it is straight, meandering, or braided. In general, braided rivers are relatively steep and meandering rivers have more gentle slopes. Meandering rivers are not subject to rapid movement and are reasonably predictable in behavior. Nevertheless they are generally unstable with eroding banks.

The Meandering stream:

A meandering stream consists of pools and crossings. The thalweg, or main current of the channel, flows from the pool through the crossings to the next pool forming the typical s-curve. In pools, the channel cross-section is somewhat triangular. Point bars form on the inside of the bends. In the crossings, the channel cross-section is more rectangular and depths are smaller. At low flows the local slope is steeper and velocities are larger in the crossing than in the pool. At low stages the thalweg is located very close to the outside of the bend.

At higher stages, the thalweg tends to straighten. More specifically, the thalweg moves away from the outside of the bend encroaching on the point bar to some degree. In general, the process of erosion and deposition forms bends. As a meandering stream moves laterally and longitudinally the meander loops move at unequal rates because of the unequal erodability of the banks. The channel geometry depends on the local slope, the bend material, and the geometry of the adjacent bends.

7.3 Alluvial Streams (continued)

7.3.1 Classification (continued)

The Meandering Stream: (continued)

A meandering river has more or less regular inflections that are sinuous in plan. It consists of a series of bends connected by crossings. In the bends, deep pools are carved adjacent to the concave banks by the relatively high velocities. On the inside of the bends, as velocities are lower, sediments are deposited, forming the point bar. Much of the sediment eroded from the outside bank is deposited in the crossing and on the point bar in the next bend downstream. The crossings are short, straight reaches which connect the bends; they are quite shallow compared to the pools in the bendways.

The geometry of meandering rivers is quantitatively measured in terms of (1) meander wavelength, l; (2) meander width, Wm; (3) mean radius of curvature, Rc; (4) mean amplitude, a; and (5) bend deflection angle.

The Braided Stream:

A braided stream is one that consists of multiple and interlacing channels. One cause of braiding is the large quantity of bed load the stream is unable to transport. Another cause of braiding is easily eroded banks. The braided stream presents difficulties because it is unstable, changes alignment rapidly, carries large quantities of sediment, is very wide and shallow even at flood flows and is, in general, unpredictable.

The excess bed load results in braiding when the channel is overloaded with sediment, deposition occurs, the bed aggrades, and the slope of the channel increases in an effort to obtain a graded state. As the channel steepens, the velocity increases, and multiple channels develop. These interlaced multiple channels cause the overall channel system to widen. The multiple channels result as bars of sediment are deposited within the main channel.

Erodible banks contribute to braiding as a response to changing flows, the stream widens at high flows and forms bars at low flow which become stabilized, forming islands. In general, a braided channel has a large slope, a large-bed material load in comparison with its suspended load, and relatively small amounts of silts and clays in the bed and banks.

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change

The major complicating factors in river mechanics are: 1) the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a stream system; and 2) the continual evolution of stream channel patterns, channel geometry, bars, and forms of bed roughness with changing water and sediment discharge. In order to better understand the responses of a stream to the actions of man and nature, a few simple hydraulic and geomorphic concepts are presented herein.

Hydraulic geometry is a general term applied to alluvial channels to denote relationships between discharge, Q, and the channel morphology, hydrology, and sediment transport. In alluvial channels, the morphologic, hydraulic and sedimentation characteristics of the channel are determined by a large variety of factors. The mechanics of such factors are not fully understood; however, alluvial streams do exhibit some quantitative hydraulic geometry relations. The hydraulic geometry relations express the integral effects of all the hydrologic, meteorological and geologic variables in a drainage basin.

In general, the hydraulic geometry relations are stated as power functions of the discharge. The dependent variables identified are the channel width, W; the channel depth, Yo; the average velocity, v; the total bed sediment load, Qt; the average friction slope, Sf; and the average Manning's roughness coefficient, n.

At a given cross-section, width, depth and velocity increase systematically with discharge. A base condition is established by the dominant discharge, usually a flow of 1.5 to 2.33 years frequency. The geologic character of the region through which the channel runs initially establishes the longitudinal slope of a channel.

The initial slope is modified to a greater or lesser extent by a stream, or by is ancestral streams. The dependence of stream form on slope, which may be imposed independently of other stream characteristics, is illustrated schematically in Figure 7-3.

Any natural or artificial change that alters channel slope can result in modifications to the existing stream pattern. For example, a cutoff of a meander loop decreases channel sinuosity and increases channel slope. Referring to Figure 7-3, this shift in the plotting position to the right could result in a shift from a relatively tranquil, meandering pattern toward a braided pattern that varies rapidly with time, has high velocities, is subdivided by sandbars, and carries relatively large quantities of sediment. Conversely, it is possible that a slight decrease in slope could change an unstable braided stream into a meandering one.

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change (continued)

The different channel dimensions, shapes, and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel morphology can be anticipated. Further, a change in hydrology may cause changes in stream sinuosity, meander wavelength, and channel width and depth. A long period of channel instability with considerable bank erosion and lateral shifting of the channel may be required for the stream to compensate for the hydrologic change.

Stream Stability Problems:

For engineering purposes, an unstable channel is one whose rate or magnitude of change is great enough to be a significant factor in the planning or maintenance of a highway crossing during the service life of the structure. The kinds of changes considered are (1.) lateral bank erosion, (2.) degradation or aggradation of the stream bed that continues progressively over a period of years; and (3.) natural short-term fluctuations of streambed elevations that are usually associated with the passage of a flood (scour and fill).

Stability is inferred mainly from the nature of point bars, the presence or absence of cut banks, and the variability of stream width.

Bank Stability

On a laterally unstable channel, or at actively migrating bends on an otherwise stable channel, the point bars are usually wide and unvegetated and the bank opposite to a point is cut and often scalloped by erosion. The crescentic scars of slumping may be visible from place to place along the bank line. The presence of a cut bank opposite to point bar is evidence of instability, even if the point bar is vegetated.

Along an unstable channel, bank erosion tends to be localized at bends, and straight reaches tend to be relatively stable. However, meandering of the thalweg in a straight reach is likely to be a precursor of instability. Bars that occur alternately from one side to the other of a straight reach are somewhat analogous to point bars and are indicative of a meandering thalweg.

Bank Erosion Rates: Although it is theoretically possible to determine bank erosion rates from factors such as water velocity and resistance of the banks to erosion, practical and accurate means of making such determination are still deficient. Past rates of erosion at a particular site provide the best estimate of future rates. In projecting past rates into the future, consideration must be given to the following factors: (1.) the past flow history of the site during the life of the highway crossing. The duration of floods, or of flows near bankfull stages, is probably more important that the magnitude of floods; and (2.) man-induced factors that are likely to affect bank erosion rates. Among the most important of these are urbanization and the clearing of flood plain forests.

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change (continued)

Behavior of meander loops

While no two meanders will behave in exactly the same way, meanders on a particular stream reach will tend to conform to one of several modes of behavior. The modes are of

A. extension	B. translation
C. rotation	D. conversion to a compound loop
E. neck cutoff by a closure	F. diagonal cutoff by a chute
	G. neck cutoff by a chute

Mode A represents the typical development of a loop of low amplitude, which decreases in radius as it extends slightly in a downstream direction. Mode B rarely occurs unless meanders are confined by valley sides on a narrow flood plain, or are confined by artificial levees.

Mode C is a pattern typically followed by well-developed meanders on streams that have unstable banks. Mode E also applies to loops on meandering or highly meandering streams, usually of the equiwidth point-bar type. The banks have been sufficiently stable for an elongated loop to form (without being cutoff), but the neck of the loop gradually being closed and cutoff will eventually occur at the neck.

Modes F and G apply mainly to locally braided sinuous or meandering streams having unstable banks. Loops are cutoff by chutes that break diagonally or directly across the neck.

Effect of meander cutoff

The cutoff of a meander will cause a local increase in channel slope, which results in an increase in the growth rate of adjoining meanders, and an increase in channel width at the point of cutoff. On a typical wide-bend point-bar stream the effects of cutoff do not extend very far upstream or downstream.

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change (continued)

Assessment of degradation.

Annual rates of degradation averaged from past records such as the closure of a dam give poor estimates of future rates of degradation. Typical situations exhibit an exponential decay function of the rate of channel degradation.

Indicators of degradation are listed in approximate order of reliability:
1. channel scarps, headcuts, and nickpoints
2. gullying of minor side tributaries
3. high and steep unvegetated banks
4. measurements of streambed elevation
5. changes in stream discharge relationships
6. measurement of longitudinal profile

Trends in response to change.

Figure 7-4 illustrates the dependence of river form on channel slope and discharge. It shows that when $SQ^{1/4} \leq .0017$ in a sandbed channel, the stream will meander. Similarly, when $SQ^{1/4} \geq .010$, the stream is braided.

In these equations, S is the channel slope in feet-per-foot and Q is the mean discharge in cfs. Between these values of $SQ^{1/4}$ is the transitional range.

Many U.S. rivers plot in this zone between the limiting curves defining meandering and braided streams. If a stream is meandering but its discharge and slope border on a boundary of the transitional zone, a relatively small increase in channel slope may cause it to change, in time, to a transitional or braided stream.

7.3 Alluvial Streams (continued)

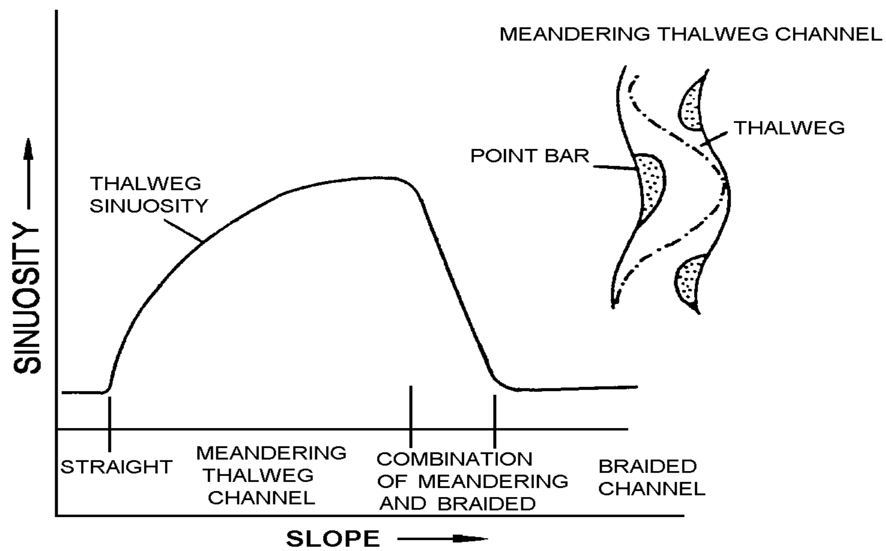


Figure 7-3 Sinuosity versus Slope with Constant Discharge
Source: After, Richardson et al., 1988

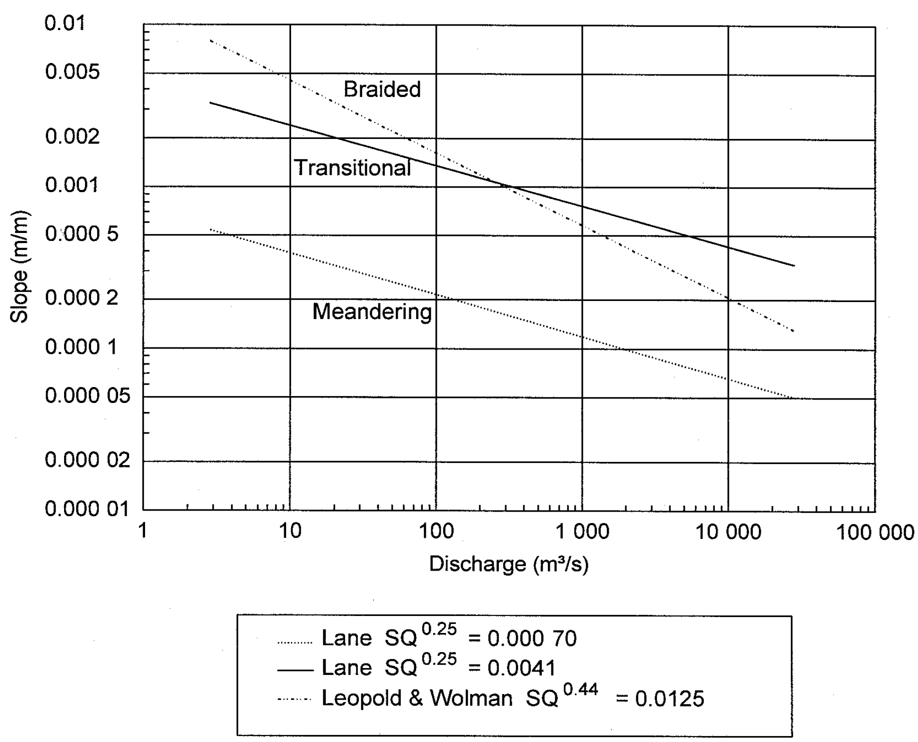


Figure 7-4 Slope-Discharge For Braiding Or Meandering Bed Streams
Source: After, Lane, 1957

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change (continued)

The resistance of banks to erosion, an important factor in stream morphology, depends on properties of the bank material and on the vegetal cover. The sediment load carried by the stream is inter-active with the longitudinal slope and the erosion resistance of the bed and bank material. The interaction often is demonstrated by the channel plan form. If a stream lacks sufficient slope to transport the material being supplied it from the drainage basin, the channel will fill until sufficient slope is attained. Attainment of the requisite slope may be accompanied by a change in plan form from sinuous to a braided pattern.

Streams having non-resistant banks, composed mainly of sand and lacking dense vegetal cover, are usually braided and have wide shallow cross-sections. Streams with banks that are resistant because of high clay content or dense vegetal cover often are meandering streams of nearly uniform width and deep, narrow cross-section.

Table 7-5 Qualitative response of alluvial channels

Variable	Change in Variable	Effect on						
		Regime of Flow	River Form	Resistance to flow	Energy Slope	Stability of channel	Area	Stage
Discharge	+	+	M-->B	+/-	-	-	+	-
	-	-	B-->M	+/-	+	+	-	+
Bed Material Size	+	-	M-->B	+	+	+/-	+	+
	-	+	B-->M	-	-	+/-	-	-
Bed Load	+	+	B-->M	-	-	+	-	-
	-	-	M-->B	+	+	-	+	+
Wash load	+	+		-	-	+/-	-	-
	-	-		+	+	+/-	+	+
Viscosity	+	+		-	-	+/-	-	-
	-	-		+	+	+/-	+	+
Seepage Force	+	-	B-->M	+	-	+	+	+
	-	+	M-->B	-	+	-	-	-
Vegetation	+	-	B-->M	+	-	+	+	+
	-	+	M-->B	-	+	-	-	-
Wind	+	+	M-->B	-	+	-	-	-
	-	-	B-->M	+	-	-	+	+

7.3 Alluvial Streams (continued)

7.3.3 Guidelines for Assessing Geomorphic Factors

1.) In evaluating a site or planning countermeasures at a bridge, look for man-induced factors that may lead to problems such as aggradation, degradation, or lateral erosion. If the stream under consideration is tributary to another stream, consider also the effects of man-induced changes on the larger stream.

2.) Compare recent and older aerial photographs. Make an assessment of stream stability and behavior.

3.) Perform a field evaluation of site. Recent aerial photographs may be used as a guide. Look particularly for indications of recent bank erosion and development of bars.

7.3.4 Countermeasures

A countermeasure is defined as a measure incorporated into a highway crossing of a stream to control, inhibit, change, delay, or minimize stream and bridge stability problems. They may be installed at the time of highway construction or retrofitted to resolve stability problems at existing crossings. Retrofitting is good economics and good engineering practice in many locations because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage, and indeed, may take a period of several years to develop. The selection of an appropriate countermeasure for a specific bank erosion problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism, and costs.

7.3.4.1 Meander Migration

The best countermeasure against meander migration is a crossing location on a relatively straight reach of the stream between bends. Other counter measures include the protection of an existing bank line, the establishment of a new flow line or alignment, and the control and constriction of channel flow. Countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads, and channel relocations. Measures may be used individually or a combination of two or more measures may be used to combat meander migration at a site (FHWA, 1990; and HEC-20, 1991).

7.3.4.2 Channel Braiding

Countermeasures used at braided streams are usually intended to confine the multiple channels to one channel. This tends to increase sediment transport capacity in the principal channel and encourage deposition in secondary channels. The measures usually consist of dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. Spur dikes at bridge ends used in combination with revetment on highway fill slopes, riprap on highway fill slopes only, and spurs arranged in the stream channels to constrict flow to one channel have also been used successfully.

7.3 Alluvial Streams (continued)

7.3.4 Countermeasures (continued)

7.3.4.3 Degradation

Degradation in streams can cause the loss of bridge piers in stream channels, and piers and abutments in caving banks. A check dam, which is a low dam or weir constructed across a channel, is one of the most successful techniques for halting degradation on small to medium streams.

Longitudinal stone dikes placed at the toe of channel banks can be effective countermeasures for bank caving in degrading streams. Precautions to prevent outflanking, such as tiebacks to the banks, may be necessary where installations are limited to the vicinity of the highway stream crossing. In general, channel lining alone is not a successful countermeasure against degradation problems (HEC-20).

7.3.4.4 Aggradation

Current measures in use to alleviate aggradation problems at highways include channelization, bridge modification, continued maintenance, or combinations of these.

Channelization may include excavating and cleaning channels, constructing cutoffs to increase the local slope, constructing flow-control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating problems with aggradation.

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels

7.3.5.1 Regimes of flow in alluvial sand bed channels

In alluvial streams and rivers, the material is eroded, moved and shaped by the flow so that the bed configuration and resistance to flow are a function of the flow and may change to increase or decrease the water surface level. The interaction between the flow of the water-sediment mixture and the sandbed creates different bed configurations that change the resistance to flow and the rate of sediment transport. The actual shape of the river is affected by numerous and interrelated variables.

The flow in alluvial sand bed channels is divisible into two regimes separated by a transition zone. Each regime is characterized by similarities in the

1.) shape of the bed configuration,

2.) mode of sediment transport,

3.) process of energy dissipation, and

4.) phase relation between the bed and water surface.

Lower flow regime	Transition	Upper flow regime
Ripples	ranges from dunes	Plane bed
Dunes with ripples superimposed or antidunes	to plane bed	Antidunes with a.) standing waves or b.) breaking
Dunes	antidunes	Chutes and pools

Lower flow regime --In the lower flow regime, resistance to flow is large and sediment transport is small. The bedform is either ripples or dunes or some combination of the two. The water-surface undulations are out of phase with the bed surface, and there is a relatively large separation zone downstream from the crest of each ripple or dune.

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels (continued)

Transition zone--The bed configuration in the transition zone is erratic, with the configuration during flow dependent mainly on the antecedent bed condition. Resistance to flow and sediment transport also have some variability. This variability during flow occurs because of the unstable state between bed form, resistance, and changes in depth and slope. Resistance to flow is small for flow over a plane bed; so the shear stress decreases and the bed form changes to dunes, the dunes cause an increase in resistance to flow which increases the shear stress in the bed and the dunes wash out, forming a plane bed, and the cycle continues.

Upper flow regime--In the upper flow regime, resistance to flow is small and sediment transport is large. The usual bed forms are plane bed or antidune. The water surface is in phase with the bed surface except when an antidune breaks, and the fluid does not normally separate from the boundary.

Resistance to flow is the result of grain roughness with the grains moving, of wave formation and subsidence, and of energy dissipation when the antidunes break.

7.3.5.2 Bed Configuration

The bed configurations (roughness elements) that commonly form in sand bed channels are:

(1) plane bed without sediment movement

(2) ripples

(3) ripples on dunes

(4) dunes

(5) plane bed with sediment movement

(6) antidunes

(7) chutes and pools

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels (continued)

These bed configurations are listed in order of their occurrence with increasing stream power for bed materials having a D50 of less than 0.6 mm. For bed materials coarser than 0.6 mm, dunes form instead of ripples after beginning of motion at small values of stream power.

The different forms of bed roughness are not mutually exclusive in time or space in a stream. Different bed-roughness elements may form side-by-side in a cross section or reach of a natural stream giving a multiple roughness; or the may form in time sequence.

Multiple roughness, the occurrence of different bed-roughness elements side-by-side in a cross-section or reach is related to variation in shear stress in a channel cross-section. The greater the width-depth ratio of a stream, the greater is the probability of a spatial variation in shear stress, stream power, or bed material. Changes in roughness over time, called variable roughness is related to changes over time in shear stress or stream power. A common example of the effect of changing shear stress or stream power is the change in bed form that occurs with changes in depth during a runoff event.

Bed configurations and their associated flow phenomena are described in the following paragraphs, in order of their occurrence with increasing stream power:

Plane bed without sediment movement

The bed configuration at levels of flow with no sediment movement is a remnant of the configuration formed when the flow was sufficient for sediment movement. Prior to beginning motion, the resistance to flow is of rigid-boundary hydraulics. After the beginning of motion, the bed configuration for flat slopes and low velocities would be either ripples, if the bed material is sand or smaller than 0.6 mm, dunes, for coarser material.

Ripples

Ripples are small triangular-shaped elements having gentle upstream slopes and steep downstream slopes. The length ranges from 0.4 to 2.0 feet and height from 0.02 ft. to 0.2 feet. The ripples result in a large resistance to flow, with Manning's "n" ranging from 0.018 to 0.030. The resistance to flow decreases as flow depth increases. The ripple shape is independent of sand size and at large values of Manning's "n", the magnitude of grain roughness is small relative to form roughness. Ripples cause very little, if any, disturbance on the water surface, and the flow contains very little suspended bed material. The bed material concentration is small, ranging from 10 to 200 ppm.

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels (continued)

Dunes

As the shear stress or the stream power is increased for a bed having ripples, or a plane bed for bed material coarser than 0.6mm, sand waves called dunes form on the bed. At smaller shear-stress values, the dunes have ripples superimposed on their backs. These ripples disappear at large shear values, particularly if the bed material is coarse sand with a D50 greater than 0.4 mm.

Dunes are triangular shaped elements similar to ripples. The length ranges from two feet to many hundreds of feet. The maximum amplitude to which dunes can develop is approximately the average depth. Hence the amplitude of dunes can increase with increasing depth of flow. With dunes, the relative roughness can remain essentially constant or even increase with increasing depth of flow. The resistance to flow is large, Manning's "n" ranges from 0.020 to 0.040. The form roughness for flow with dunes is equal to or larger than sand grain roughness.

Field observations indicate that dunes can form in any sand channel, irrespective of the size of the bed material, if the stream power is sufficiently large to cause transport of the bed material without exceeding a Froude number of unity. Dunes result in boils forming on the surface of the stream. The boils are a reflection of the large separation of the flow caused by the dunes. The water surface is out-of-phase with the bed surface.

Plane bed with movement

As the stream power of the flow increases further, the dunes elongate and reduce in amplitude. This bed configuration is called the transition or washed-out dunes. The next bed configuration is plane bed with movement. With coarse sands, larger slopes are required to affect the change from transition to a plane bed and result in larger velocities and larger Froude numbers. Manning's "n" for plane bed sand channels with sediment movement ranges from 0.010 to 0.013.

Antidunes

Antidunes are physically identical to dunes, however they may move upstream as well as downstream, or stay stationary. Also the water surface waves are in phase with the bed waves. Resistance to flow of antidunes depends on how often the antidunes form, the area of the stream they occupy, and the violence and frequency of their breaking. If the antidunes do not break, resistance to flow is about the same as that for flow over a plane bed. If many antidunes break, resistance to flow is larger, because the breaking waves dissipate a considerable amount of energy. With breaking waves, Manning's "n" varies from 0.012 to 0.02

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels (continued)

Chutes and pools

At very steep slopes, alluvial channels flow changes to chutes and pools. The resistance to flow may be very large, with Manning's "n" of 0.018 to 0.035.

Bars

In natural channels, some other bed configurations are also found. Bars are bed forms having lengths of the same order as the channel width or greater and heights comparable to the mean depth of the generating flow. Several different types of bars are observable. They are classified as:

a.) Point bars which occur adjacent to the convex banks of channel bends. Their shape may vary with changing flow conditions and motion of bed particles, but they do not move relative to the bend.

b.) Alternate bars which occur in somewhat straighter reaches of channels and tend to be distributed periodically along the reach, with consecutive bars on opposite sides of the channel. Their lateral extent is significantly less than the channel width. Alternate bars may move slowly downstream.

c.) Transverse bars may also occur in straight channels. They occupy nearly the full channel width. They occur both as isolated and periodic forms along a channel, and move slowly downstream.

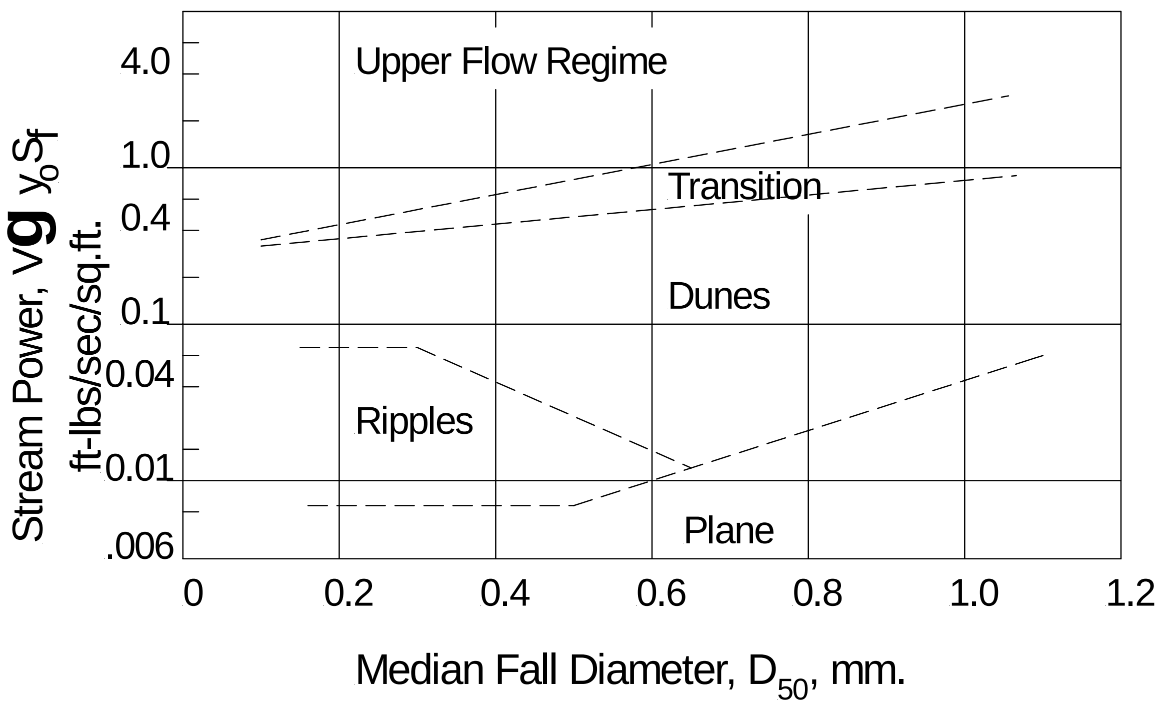
d.) Tributary bars occur immediately downstream from points of lateral inflow into a channel.

In longitudinal section, bar are approximately triangular, with very long gentle upstream slope and short downstream slopes of approximately the same as the angle of repose. Bars appear as small barren islands during low flows. Portions of upstream slope of bars are often covered with ripples or dunes.

7.3 Alluvial Streams (continued)

7.3.6 Resistance to Flow in Alluvial Channels

Resistance to flow in alluvial channels is a complex reaction affected by a large number of factors and by the interdependency of these factors. Some of the variables that describe alluvial channel flow are: velocity, depth, slope of the energy-grade line, density of water-sediment mixture, gravitational acceleration, fall diameter of the bed material, density of the sediment, shape factors of the particles, bed and fine material concentrations and the critical shear stress. The relation between stream power, median fall diameter of bed material, and form roughness is shown in figure 7.5. The relationship gives an indication of the form of the bed roughness one can anticipate if the stream power and fall diameter of bed material are known.



Lower Flow Regime	
Ripples	0.018<= n <= 0.028
Dunes	0.020<= n <= 0.040
Upper Flow Regime	
Plane Bed	0.010 <= n <= 0.013
Antidunes	
Standing Waves	0.010 <= n <= 0.015
Breaking Waves	0.012 <= n <= 0.020
Chute and Pools	0.018 <= n <= 0.035

Figure 7-5 Stream Power vs. Fall Diameter

7.3 Alluvial Streams (continued)

7.3.6 Resistance to Flow in Alluvial Channels (continued)

Bed form changes affect the impact of the stream on its boundaries. At high flows, most sandbed channel streams shift from a dune bed to a transition or a plane bed configuration. The resistance to flow is then decreased two or three fold. The corresponding increase in velocity can increase scour around bridge piers, abutments, spur dikes, or banks and increase the size of the scour protection required. Conversely an increase in resistance can result in an increase in flow depth, requiring an increase in elevation of the bridge crossing, the height of embankments, the height of any dikes, and the height of any channel control works.

A very important effect of bed forms and bars is the change in flow direction in channels. At low flows, the bars can be residual and cause high-velocity flow along or at a pier or abutment or any other structure in the streambed, causing deeper than anticipated scour. An effect of dunes on the bed is a fluctuating pattern of scour on the bed and around piers, abutments, guide banks, and spur dikes. The average height of dunes is 1/2 to 1/3 the average depth of flow, and the maximum height of a dune may approach the average depth of flow.

When analyzing the sandbed river environment, care must be used in analyzing the crossing in order to foresee possible changes that may occur in the bed form and what this change may do to the resistance coefficient, to sediment transport, and to the stability of the reach and its structures.

Alluvial processes and resistance to flow in coarse material streams

The behavior of coarse material channels is somewhat different from the sandbed channels. This includes all channels with non-cohesive bed material coarser than 2 mm size. In general, the coarse material channels are less active and slower in bank shifting than the sandbed channels. Often the bed material in the mobile bed channel is rearranged during flow resulting in armoring. The armoring phenomenon is a covering of the bed by a one particle thick layer of the coarser material underlain by the finer sizes. The absence of finer sizes from the surface layer is caused by the winnowing away of these sizes by the flows. As the range of particle sizes available in the bed of coarse-material is large, these channels can armor their beds and behave as rigid boundary channels for flows below the armoring event. The bed and bank forming activity in these channels is therefore limited to much smaller intervals of the annual hydrographs than the sandbed channels.

The general lack of mobility in coarse material channels also means the bed forms do not change as much or as rapidly as in sandbed channels. The roughness characteristics of coarse material channels are more consistent during the annual hydrographs than sandbed channels. The major component of the resistance to flow in coarse material channels comes from grain roughness and from bars; bed forms (dunes) are a lesser factor in flow resistance. Ripples never form and dunes are rare. The main type of form roughness is the pool and rifle configuration. With this type of configuration, the grain roughness is the main component of the channel roughness.

7.3 Alluvial Streams (continued)

7.3.7 Sediment Motion

The beginning and ceasing of sediment motion can be related to either the shear stress on the grains or to the fluid velocity in the vicinity of the grains. When the grains are at incipient motion, these values are called the critical stress or critical velocity.

The forces acting on an individual particle on the bed of an alluvial channel are:

1.) the body forces due to gravity,
2.) the external forces acting at the points of contact between the grain and its neighboring grains, and
3.) the fluid forces acting on the surface of the grain.

The fluid force varies with the velocity field and with the properties of the fluid. The relative magnitude of these forces determines whether the grain moves or not. The fluid forces may be divided into three components: 1.) form drag, 2.) viscous drag, and 3.) buoyant force. The form drag can be expressed in terms of the shear velocity. The viscous drag is related to the shear velocity for laminar flow. When flow over a grain is turbulent, the form drag is predominant; when the flow is laminar, the viscous shear force is predominant. In view of the bed consisting of particles of various sizes, each one having different shear stresses needed to dislodge, it is customary to consider the critical shear stress corresponding to the D50 size of the bed material as that required for beginning of motion.

7.3.8 Sediment Transport

The amount of material transported or deposited in the stream under a given set of conditions is the result of the interaction of two groups of variables. In the first group are those variables which influence the quantity and quality of the sediment brought down to that section of the stream. In the second group are variables that influence the capacity of the stream to transport that sediment.

Group I. - Sediment brought down to the stream depends on the geology and topography of the watershed; magnitude, intensity, duration, distribution, and season of rainfall, soil condition, vegetal cover, cultivation and grazing, surface erosion, and bank cutting.

Group II. - Capacity of the stream to transport sediment depends on hydraulic properties of the stream channel. These are fluid properties, slope, roughness, hydraulic radius, discharge, velocity, velocity distribution, turbulence, tractive effort, viscosity and density of the fluid, sediment mixture, and size and gradation of the sediment.

These variables are not all independent of each other, nor is their effect, in some cases, definitely known. The variables which control the amount of sediment brought down to the stream are subject to so much variation, not only between streams but at a given point of a single stream, that the analysis of any particular case in a quantitative way is extremely difficult. The variables that deal with the capacity of the stream to transport solids are subject to mathematical analysis. These variables are closely related to the hydraulic variables controlling the capacity of the streams to carry water.

7.3 Alluvial Streams (continued)

7.3.8 Sediment Transport (continued)

Sources of Sediment Transported

For engineering purposes, there are two sources of sediment transported by a stream: (1.) the bed material that make up the streambed; and (2) the fine material that comes from the banks and the watershed (wash load). Geologically both materials come from the watershed. But the distinction is important because the bed material is transported at the capacity of the stream and is functionally related to the measurable hydraulic variables. The wash load is not transported at the capacity of the stream, it is dependent on the availability and is not functionally related to measurable hydraulic variables.

Total Sediment Discharge

The total sediment discharge of a stream is the sum of the bed sediment discharge and the fine sediment (wash load) discharge, or the sum of the contact sediment discharge and suspended sediment discharge. In the former sum the total sediment discharge is based on the sources of the sediments and the latter sum is based on the mode of sediment transport. The suspended sediment load consists of both bed sediment and fine sediment (wash load), only the bed sediment discharge can be estimated by the various equations that have been developed.

There are many equations developed for the estimation of bed sediment transport. The variation between the magnitudes of the bed sediment discharge predicted by the different equations under the same conditions is tremendous. For the same discharge, the predicted sediment discharge can vary by a 100-fold difference between the smallest and the largest discharge. This can be expected given the number of variables, the interrelationships between them, the difficulty of measuring many of the variables and the statistical nature of bed material transport.

Suspended bed sediment discharge is usually computed by one of three methods. These are Meyer-Peter Muller (1948), Einstein (1950), and Colby (1961). The Meyer-Peter Muller equation is applicable to streams with little or no suspended sediment discharge and is thus used extensively for gravel and cobble bed streams. The other two methods, based to some degree on Einstein's work are used for sandbed channels.

7.3 Alluvial Streams (continued)

7.3.8 Sediment Transport (continued)

Sediment transport in coarse material channels.

The bed material load in coarse-bed channels is mostly transported as bed load and not as suspended load. For the bed-load transport, Einstein's bed-load function (without the suspended load component) and the Meyer-Peter Muller transport function may be found useful.

The time response of coarse-material channels is different from sandbed channels in the time scale of response. This time response is dominated by two factors: (1.) the difference in particle size between the surface (armor) layer of the bed and the bed material below it; and, (2) the wash load may extend to coarse sand sizes.

The formation of an armor layer on the bed may immobilize the bed for a large part of the hydrograph. However, if the conditions for incipient movement of this layer are exceeded, the underlying fine bed material will be readily picked up by the flow. Thus, extreme flow events in coarse-material channels are capable of inducing rapid and large bed-level fluctuations.

The coarse sand and larger particles may behave as wash load in coarse material channels; that is, although the flow may be transporting a large quantity of these particles, the boundary shear may be large, so that these particles are not found in appreciable quantities in the armor layer. If the boundary shear is reduced by afflux at a highway crossing, the flow may not sustain this material as wash load and rapid aggradation may occur.

When considerable constriction is imposed at a bridge crossing by the bridge approach or river training works sediment problems should be anticipated.

7.4 Principles of Open Channel Flow

7.4.1 General

Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow (see Chow, 1970; Henderson, 1966). The basic principles of fluid mechanics -- continuity, momentum, and energy -- are applicable to open channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principle problems of open channel flow analysis and it depends on quantification of the flow resistance. Natural channels display a much wider range of roughness values than artificial channels. This section presents the application of Manning's equation and the energy equation for the computation of channel capacity.

7.4.2 Flow Classifications

7.4.2.1 Flow Classification and Determination

The classification of open channel flow can be summarized as follows.

Steady Flow

1. Uniform Flow
2. Nonuniform Flow
    a. Gradually Varied Flow
    b. Rapidly Varied Flow

Unsteady Flow

1. Unsteady Uniform Flow (rare)
2. Unsteady Nonuniform Flow
    a. Gradually Varied Unsteady Flow
    b. Rapidly Varied Unsteady Flow

The steady uniform flow case and the steady nonuniform flow case are the most fundamental types of flow treated in highway engineering hydraulics.

Steady and Unsteady Flow

Steady flow is one in which the discharge and other hydraulic properties passing a given cross-section are constant with respect to the time interval under consideration. At a section, the depth of flow does not change during the time interval under consideration. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. The flow is unsteady when the discharge and therefore depth and velocity vary with time.

7.4 Open Channel Flow (continued)

7.4.2.1 Flow Classification and Determination (continued)

Uniform Flow and Non-uniform (Varied) Flow

For uniform flow, the depth and velocity remain constant along the length of a channel, while for non-uniform (varied) flow the velocity and depth vary along the direction of flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross section, roughness, and slope along the flow direction; however, non-uniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

Gradually-varied and Rapidly-Varied

Gradually varied, non-uniform flow is one in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected. It is considered to be rapidly varied flow when the changes in velocity should not be neglected.

Steady, uniform flow is an idealized concept of open channel flow that seldom occurs in natural channels. However, for most practical highway drainage applications, the flow is steady and changes in width, depth, or direction are sufficiently small that flow can be considered uniform.

Continuity Equation

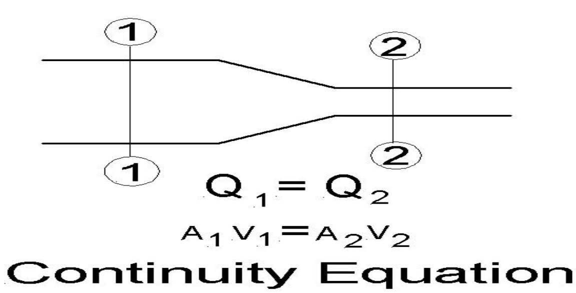
The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the simple form:

$$\mathbf{Q = A_1V_1 = A_2V_2} \qquad \mathbf{(7.4.1)}$$

Where:

Q = discharge, cfs
A = flow cross-sectional area, ft^2
V = mean cross-sectional velocity, ft/s (which is perpendicular to the cross section)

The subscripts 1 and 2 refer to successive cross sections along the flow path.



7.4 Open Channel Flow(continued)

7.4.2.1 Flow Classification and Determination (continued)

Manning's Equation

For a given channel geometry, slope, and roughness, and a specified value of discharge Q, a unique value of depth occurs in steady uniform flow. It is called the normal depth. It is computed indirectly from Manning's equation:

$$\mathbf{Q = (1.486/n)AR^{2/3}S^{1/2}} \tag{7.4.2}$$

For a given depth of flow in a uniform channel, the mean velocity is computed using the Manning's equation. Dividing the above equation by the area resulting in:

$$\mathbf{V = Q/A = (1.486/n)R^{2/3}S^{1/2}} \tag{7.4.3}$$

Where:

Q = discharge, cfs
V = mean velocity, ft/sec
n = Manning's roughness coefficient
A = cross-sectional area of flow, ft^2
R = hydraulic radius = A/P, ft
P = wetted perimeter of flow area, ft
S = channel bed slope, ft/ft

The selection of Manning's "n" is generally based on observation; however, considerable experience is essential in selecting appropriate n values. The range of 'n' values for various types of channels and floodplains is given in Appendix 7-A, Table 7-1. R, the hydraulic radius is the ratio of flow area to the wetted perimeter. The wetted perimeter is the length along the channel cross-section where the water is in contact with the channel boundaries. It is a term that gives an indication of the hydraulic efficiency of the channel.

In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance K:

$$\mathbf{K = (1.486/n)AR^{2/3}} \tag{7.4.4}$$

then Manning's Equation can be written as:

$$\mathbf{Q = KS^{1/2}} \tag{7.4.5}$$

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry and roughness characteristics alone and is independent of the streambed slope. The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross-section and the flow distribution through the opening in a proposed stream crossing.

7.4 Open Channel Flow (continued)

7.4.2.1 Flow Classification and Determination (continued)

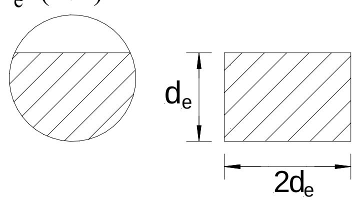
Normal Depth

In steady uniform flow for a given channel geometry, slope, and roughness, and discharge Q a unique value of depth occurs, it is called the normal depth.

Equivalent Depth

For sections that are not rectangular, it is often useful to determine hydraulic properties based on an equivalent depth of flow in a rectangular section that has a width equal to twice the depth.

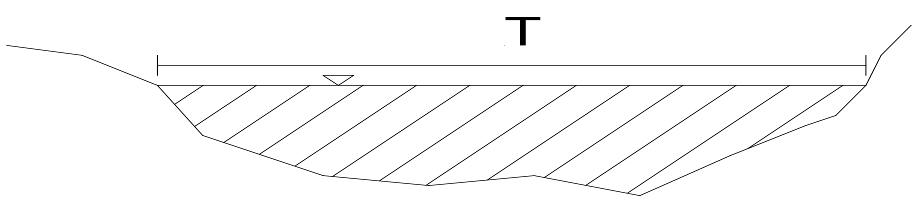
$$d_e=(A/2)^{0.5}$$



Mean Depth

The mean depth, sometimes referred to as hydraulic depth, is equal to the area of flow divided by the top width.

$$d_m={}^{A}/_{T}$$



7.4.3 Energy Principles

Total Energy Head, Energy Grade Line

Flowing water contains energy in two forms, potential and kinetic. The potential energy at a particular point is represented by the depth of the water plus the elevation of the channel bottom above a datum (elevation head). The plot of the potential energy head from one cross section to the next defines the hydraulic grade line. For open channel flow, the hydraulic grade line is coincident with the water surface. The kinetic energy is represented by the velocity head. The total energy head is the sum of potential energy head and kinetic energy head (velocity head). The plot of the total energy head from one cross section to the next defines the energy grade line.

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Energy Equation

Written between an upstream open channel cross section designated 1 and a downstream cross section designated 2, the energy equation is:

$$\mathbf{h_1 + á_1(V_1^2/2g) = h_2 + á_2(V^2_2/2g) + h_L} \qquad \mathbf{(7.4.6)}$$

Where:

h_1 and h_2 are the upstream and downstream stages, respectively, ft
á = kinetic energy correction coefficient
V = mean velocity, ft/s
h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance, ft

The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow y, i.e. h=z+y. The terms in the energy equation are illustrated graphically in Figure 7-6. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected

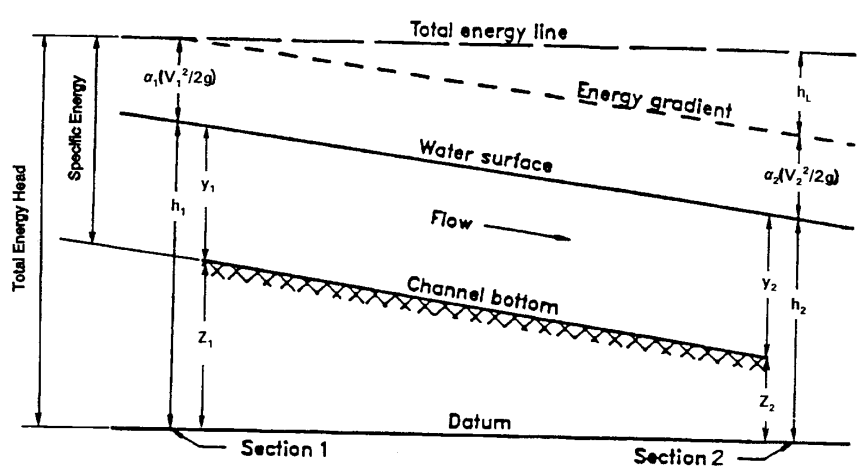


Figure 7-6 Terms In The Energy Equation

Source: FHWA, 1990

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Specific Energy

In open channel flow, it is often desirable to consider the energy content with regards to the channel bottom. Specific energy E is defined as the energy head relative to the channel bottom. If the channel is not too steep (slope less than 10 percent) and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy E becomes the sum of the depth and velocity head:

$$\mathbf{E = y + á\,(V^2/2g)} \qquad \textbf{(7.4.7)}$$

Where:

y = depth, ft
á = kinetic energy correction coefficient
V = mean velocity, ft/s
g = gravitational acceleration, 32.2 ft/sec^2

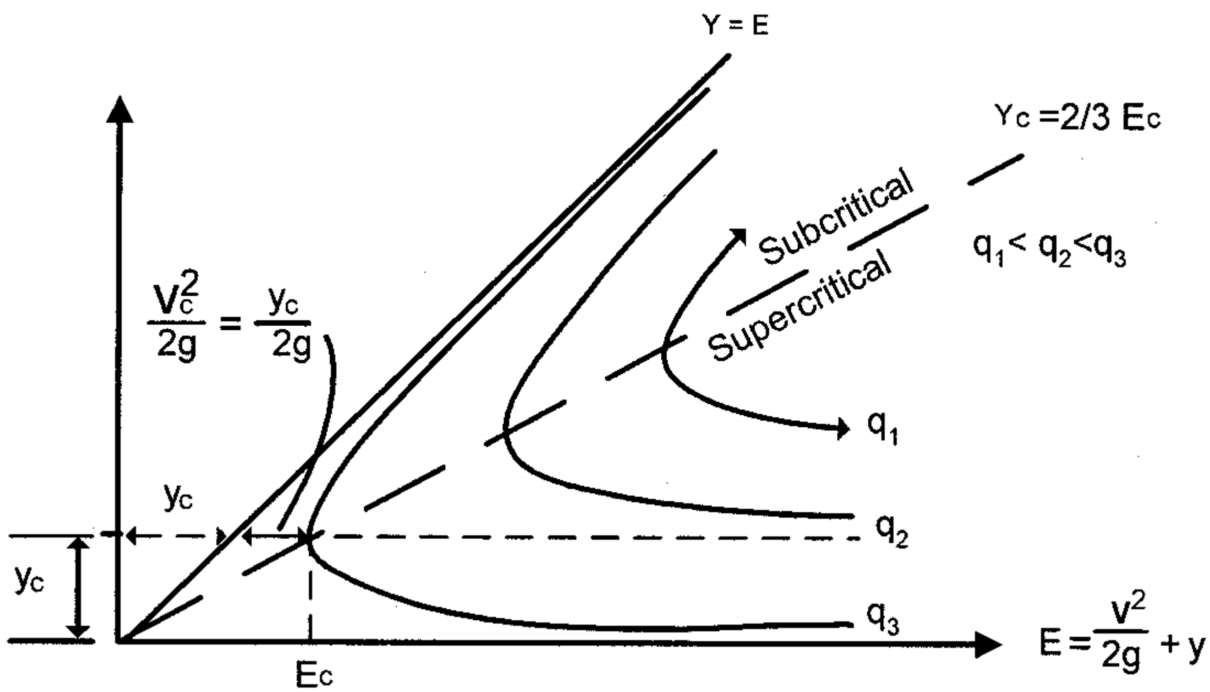


Figure 7-7 Specific Energy And Discharge Diagram For Rectangular Channels
(Adopted From Highways In The River Environment)

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Kinetic Energy Coefficient

The kinetic energy correction coefficient is taken to have a value of one for turbulent flow in prismatic channels but may be significantly different than one in natural channels. As the velocity distribution in a river varies from a maximum at the design portion of the channel to essentially zero along the banks, the average velocity head, computed as $(Q/A)^2/2g$ for the stream at a section, does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head, above, by a kinetic energy coefficient, á, defined as:

$$\alpha = [\Sigma (qv^2)/(QV^2)] \tag{7.4.8}$$

Where:
- v = average velocity in subsection, ft/sec
- q = discharge in same subsection, cfs
- Q = total discharge in river, cfs
- V = average velocity in river at section or Q/A, ft/sec

Froude Number

The Froude number is an important dimensionless parameter in open channel flow. It represents the ratio of inertia forces to gravity forces and is defined by:

$$F_r = V/(gd)^{0.5} \tag{7.4.9}$$

Where:
- V = average velocity = Q/A, ft/sec
- g = acceleration of gravity, ft/sec^2
- d = hydraulic depth, also referred to as the mean depth, d=A/T
- A = cross-sectional area of flow, ft^2
- T = channel top width at the water surface, ft

This expression for Froude number applies to any single section channel of nonrectangular shape.

The Froude number may be used to distinguish between subcritical flow and supercritical flow.

F=1, Critical Flow
F<1, Subcritical Flow
F>1, Supercritical Flow

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Critical Depth

The variation of specific energy with depth at a constant discharge shows a minimum in the specific energy at a depth called critical depth at which the Froude number has a value of one. Critical depth is also the depth of maximum discharge when the specific energy is held constant. If the normal depth computed from Manning's equation is greater than critical depth, the slope is classified as a mild slope, while on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope. These relationships are illustrated in Figure 7-8.

Subcritical Flow

Depths greater than critical occur in subcritical flow and the Froude number is less than one. Slopes are considered mild. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the control is always located downstream.

Supercritical Flow

Depths less than critical depth occur in supercritical flow and the Froude number is greater than one. Slopes are considered steep. Small water surface disturbances are always swept downstream in supercritical flow, and the location of the flow control is always upstream.

For the two types of flow, the depth, velocity, and slope characteristics vary as shown below.

	Subcritical Flow	**Supercritical Flow**
Depth	Relatively Deep	Shallow Flow
Velocity	Low Velocity	High Velocity
Slope	Mild Slope	Steep Slope

Changes in slope will result in water surface transitions. These changes can be smooth or abrupt depending on the magnitude and direction of change. The changing water surface can be classified based on the bed slope and direction of change in the depth. Figure 7-9 shows the classification of water surface profiles.

Control Section

Any cross-section for which the depth of flow can be uniquely predicted for a given discharge, most commonly at critical depth.

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

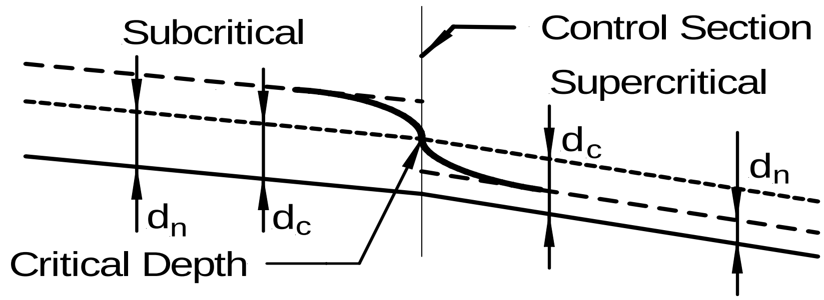


Fig. 7-8 Control Section

Table 7-6
Flow Profile Types

Classification	Class	Bed Slope	Depth	Type
M1	Mild	$S_0>0$, $S_0<S_c$	$Y>y_o>y_c$	1
M2	Mild	$S_0>0$, $S_0<S_c$	$y_o>y>y_c$	2
M3	Mild	$S_0>0$, $S_0<S_c$	$y>y_c>y_o$	3
C1	Critical	$S_0>0$, $S_0=S_c$	$y>y_o=y_c$	1
C3	Critical	$S_0>0$, $S_0=S_c$	$y<y_o=y_c$	3
S1	Steep	$S_0>0$, $S_0>S_c$	$y>y_c>y_o$	1
S2	Steep	$S_0>0$, $S_0>S_c$	$y_c>y>y_o$	2
S3	Steep	$S_0>0$, $S_0>S_c$	$y_c>y_o>y$	2
H2	Horizontal	$S_0=0$	$y=y_o>y_c$	2
H3	Horizontal	$S_0=0$	$y_c>y=y_o$	3
A2	Adverse	$S_0<0$	$y>y_o>y_c$	2
A3	Adverse	$S_0<0$	$y_c>y>y_o$	3

With a type 1 curve (M1,S1,C1), the actual depth of flow y is greater than both the normal depth y_o and the critical depth, y_c. Because flow is tranquil, control of the flow is downstream.

With a type 2 curve (M2,S2,A2, H2), the actual depth y is between the normal depth y_o and the critical depth y_c. The flow is tranquil for M2, A2, and H2 and thus the control is downstream. Flow is rapid for S2 and the control is upstream.

With a type 3 curve (M3, S3, C3, A3, and H3), the actual depth y is smaller than both the normal y_o depth and the critical depth y_c. Because the flow is rapid, the hydraulic control is upstream.

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

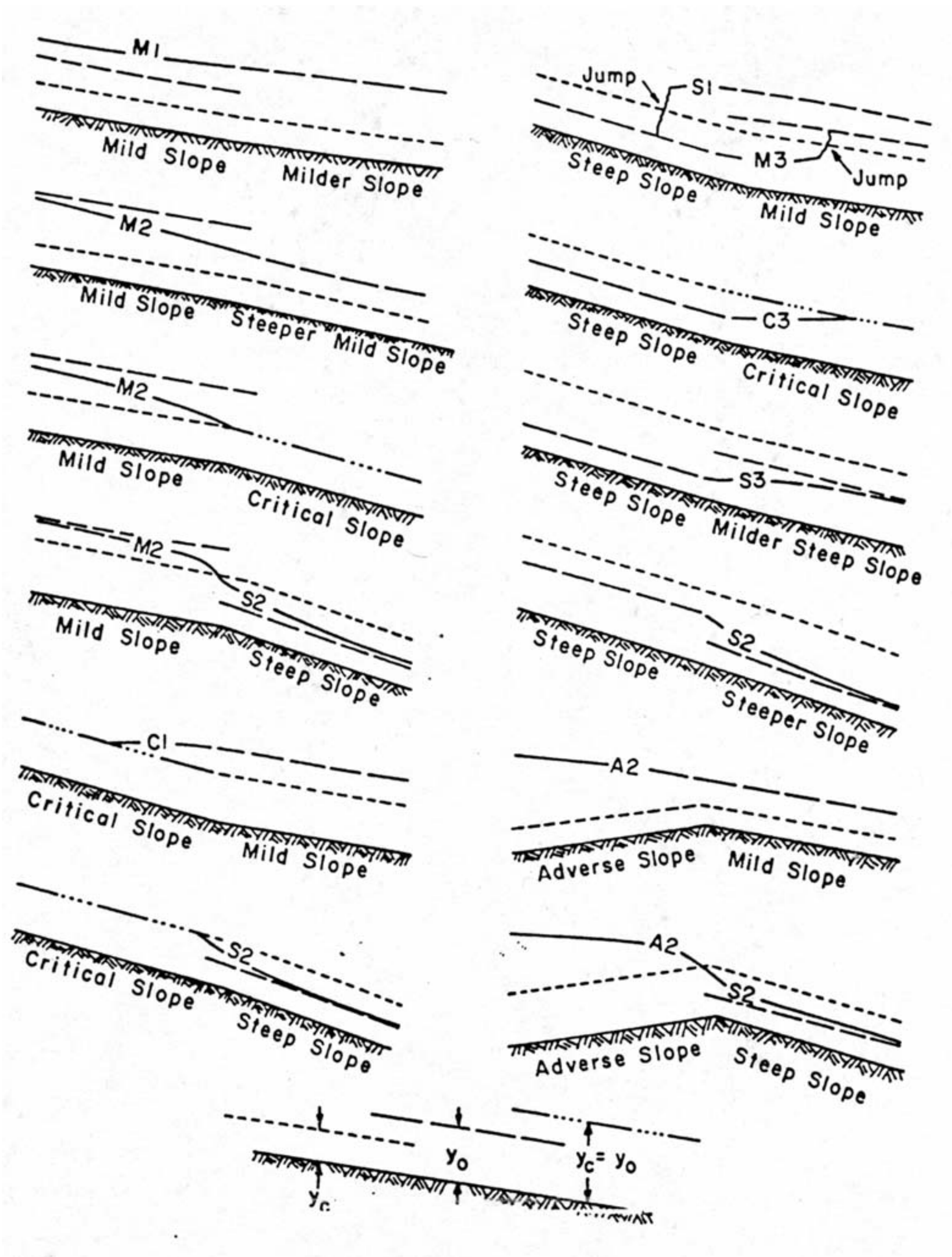


Figure 7-9 Water Surface Profiles

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Hydraulic Jump

A hydraulic jump occurs as an abrupt transition from supercritical to subcritical flow in the flow direction. There are significant changes in depth and velocity in the jump, and energy is dissipated. For this reason, the hydraulic jump is often employed to dissipate energy and control erosion at highway drainage structures. See Figure 7-10 for a plot of the hydraulic jump diagram. The two depths associated with a hydraulic jump are related by the Momentum equation.

A hydraulic jump will not occur until the ratio of the flow depth d_1 in the approach channel to the depth d_2 in the downstream channel reaches a specific value that depends on the channel geometry. The depth before the jump is called the initial depth, d_1 and the depth after the jump is the sequent depth, d_2. When a hydraulic jump is used as an energy dissipator, controls to create sufficient tailwater depth are often necessary to control the location of the jump and to ensure that a jump will occur over the desired range of discharges. Sills can be used to control a hydraulic jump if the tailwater is less than the sequent depth. (See Chow,1970)

Momentum Equation

$$\mathbf{d_2/d_1=(\{1+8*F_r^{\,2}\}^{0.5}-1)^{1/2}}$$

Figure 7-10 Hydraulic jump diagram.

7.5 Hydraulic Analysis

7.5.1 General

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness, and slope. The depth and velocity of flow are necessary for the design or analysis of channel linings and highway drainage structures.

Two methods are commonly used in hydraulic analysis of open channels. The single-section method is a simple application of Manning's Equation to analyze situations in which uniform or nearly uniform flow conditions exist, such as tailwater rating curves for culverts. The step-backwater method is used to compute the complete water surface profile, such as for bridge hydraulics, in a stream reach to evaluate the unrestricted water surface elevations, or to analyze other gradually varied flow problems in streams.

The single-section method will generally yield less reliable results because it requires more judgment and assumptions than the step-backwater method. In many situations, however, the single-section method is all that is justified, e.g., a standard roadway ditch, culverts, storm drain outfalls, etc. The selection of the method of analysis is based on the cost of application and the risk and consequences associated with the feature/element under consideration.

7.5.2 Cross Sections

The step-backwater analysis should be used where evaluation of the impact of the project is needed upstream or downstream. Cross-sections shall extend sufficiently in either direction so that all impacts are identified.

Cross-sectional geometry of streams is defined by coordinates of lateral distance and ground elevation that locate individual ground points. The cross section is taken normal to the flow direction along a single straight line where possible, but in wide floodplains or bends it may be necessary to use a section along intersecting straight lines, i.e. a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors.

Cross sections should be located to be representative of the subreaches between them. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross-sections taken at shorter intervals in order to better model the change in conveyance.

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as in the case of overbank flows. The conveyances of each subsection are computed separately to determine the flow distribution and á, and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection (Davidian, 1984). Selection of cross sections and vertical subdivision of a cross section are shown in Figure 7-11.

7.5 Hydraulic Analysis (continued)

7.5.2 Cross Sections (continued)

7.5.2.1 Manning's n Value Selection

Manning's ‘n’ is affected by many factors and its selection in natural channels depends heavily on engineering experience. Resistance to flow depends on a number of factors such as bed material size, changes in channel geometry, bed forms (dunes, ripples, etc.), and vegetation type and density. Roughness will also vary with depth, decreasing as the water depth becomes much greater than the roughness elements (vegetation, bed material, bed forms, etc). Therefore the resistance to flow will vary from season to season and year to year. Because changes will occur over time, a range of roughness values should be considered and a sensitivity analysis is recommended to identify how uncertainty in roughness value affects the computed water surface elevation and/or velocity. Consideration needs to be given to the overall goal of the model. When velocity is a critical parameter (bank protection design), a roughness value on the lower end of the range should be used, and when the water surface elevation is more critical (levee design), a higher roughness value should be used.

Pictures of channels and flood plains for which the discharge has been measured and Manning's n has been calculated are very useful (see Arcement and Schneider(1984); Barnes, 1978, ADOT, MCFCD). For situations lying outside the engineer's experience, a more regimented approach is presented in Arcement and Schneider, (1984). Once the Manning's n values have been selected, it is highly recommended that they be verified with historical high-water marks and/or gaged streamflow data.

Manning's n values for artificial channels are more easily defined than for natural stream channels. See Table 7-1 in Appendix 7-A for typical n values of both artificial channels and natural stream channels.
The following publications provide information specific to Arizona streams:

- Estimated Manning’s Roughness Coefficients for Stream Channels and Floodplains in Maricopa County, Arizona (Thomsen and Hjalmarson, 1991)
- Roughness Coefficients for Stream Channels in Arizona (Albridge and Garret, 1973)
- Verification of Roughness Coefficients for Selected Natural and Constructed Stream Channels in Arizona (Philips and Ingersoll, 1998)

In dealing with vegetated flood control channels, the modeler must account for fully vegetated conditions. One reference specific to Arizona watercourses is “Method to Estimate Effects of Flow-Induced Vegetation Changes on Channel Conveyances of Streams in Central Arizona” (Phillips et al., 1998).

The designer must exercise caution when using Manning’s n based on field reconnaissance. When the modeler inspects a watercourse to judge its roughness, the physical state of the system when it is inspected is not necessarily the physical state the system would be under at design conditions. When the design condition is a rare event, the roughness of the watercourse during such an event may be drastically different than what is seen in the field. When roughness is based upon vegetative resistance, a shear stress analysis should be conducted to check if the vegetation critical

7.5 Hydraulic Analysis (continued)

7.5.2.1 Manning's n Value Selection (continued)

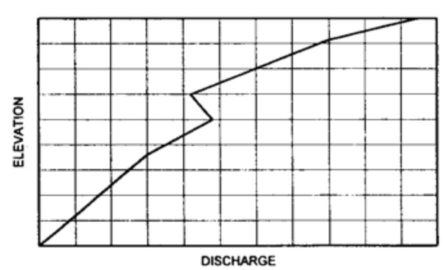
shear stress is exceeded by the actual shear stress. Conversely, if a plain streambed is observed and only the grain resistance is used to obtain the “n” value, the actual resistance may be higher under design floods because dunes or ripples may form and become important additional components in determining Manning’s n.

7.5.2.2 Calibration

The values used in the equations should be calibrated to ensure that they accurately represent local channel conditions. The following parameters, in order of preference, should be used for calibrations: Manning's n, slope, discharge, and cross-section. Proper calibration is essential if accurate results are to be obtained. Calibration is not easy, or always attainable.

7.5.2.3 Switchback Phenomenon

If the cross-section is improperly subdivided, the mathematics of the Manning's Equation causes a switchback. A switchback results when the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area that causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed which is lower than the discharge based upon the lower water depth. More subdivisions within such cross-sections should be used in order to avoid the switchback.



SWITCHBACK

7.5 Hydraulic Analysis (continued)

7.5.2 Cross Sections (continued)

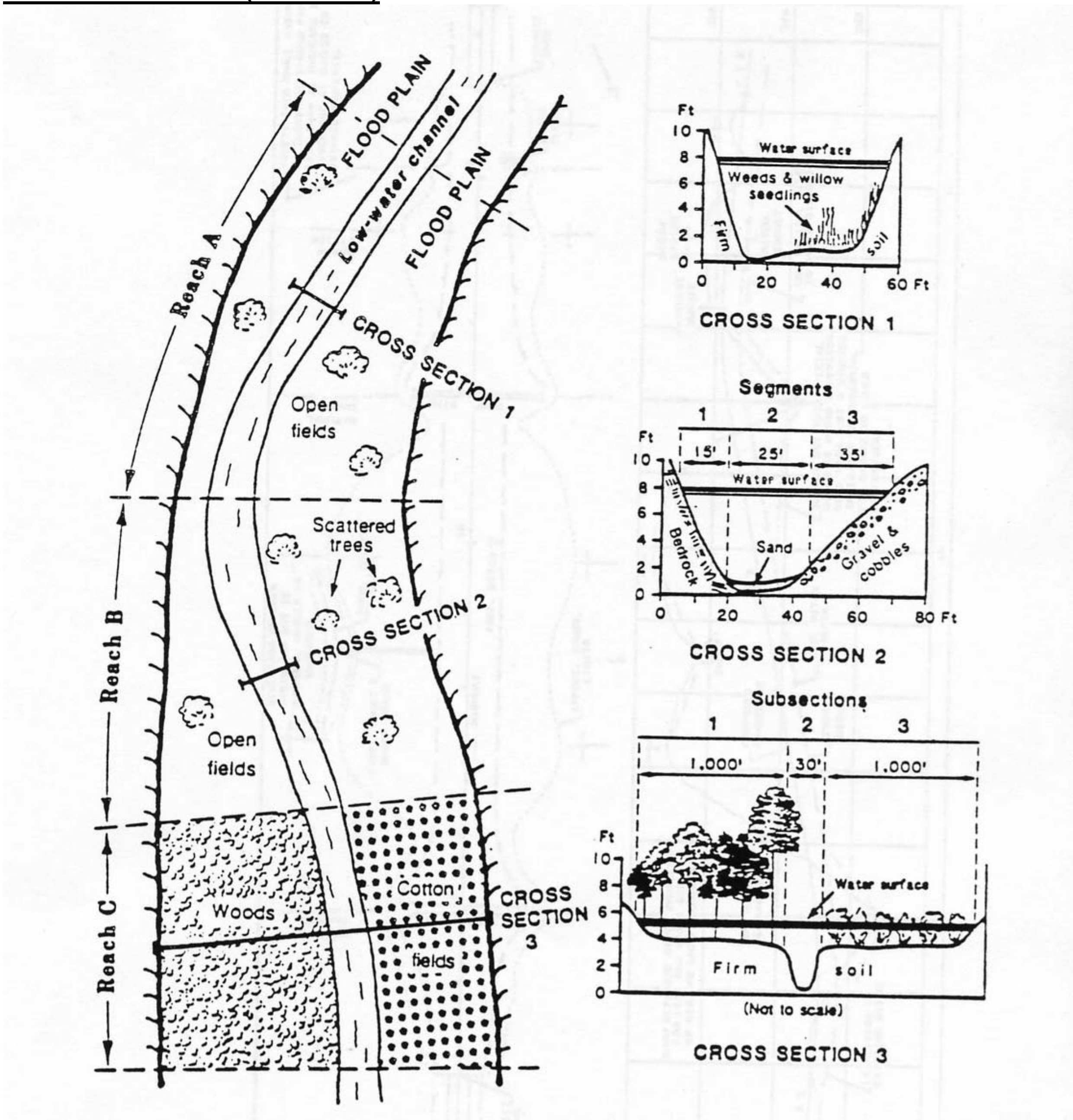


Figure 7-11 Hypothetical Cross Section Showing Reaches, Segments, And Subsections Used In Assigning n Values

Source: FHWA, 1984

7.5 Hydraulic Analysis (continued)

7.5.2.3 Switchback Phenomenon (continued)

This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross-section being used in a step-backwater program. For this reason, the cross-section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n-value, itself, may be the same in adjacent subsections.

7.5.2.4 Bends

Bends are not usually directly considered in the analysis of water surface. For bends with a radius of curvature to active channel width less than 10, Cowan (1956) presents suggestions on adjusting the Manning's n. Changing the expansion and contraction coefficients as a means to account for bends is not appropriate. A superelevated water surface should be considered for artificial channels and levees and for determining increased shear stresses on stream bank protection measures.

$$\text{Delta } S = \frac{V^2 w}{g R_c}$$

where S = superelevation, ft.

V = mean channel velocity, ft/sec

W = top channel width, ft

G = gravity, 32,2 ft/sec^2

R_c = radius of centerline of channel, ft.

Curved alignment,

Subcritical: $R_c > 3w$, If $R_c > 10w$ consider as straight.

Supercritical: $R_c > \frac{4(V^2 w)}{gy}$

7.5.2.5 Modeling of depressions (Pit Areas and Sand & Gravel Mining)

Natural or man-made depressions can be situated in-channel or off-channel. One cause may be sand and gravel mining extraction operations. For modeling pits, an appropriate number of cross-sections should be selected to properly represent the change in pit dimensions. At a minimum, there should be a cross-section at the beginning and end of the pit with an additional cross-section located at the widest part of the pit. The pit will then be represented as a diamond shape in planform.

7.5 Hydraulic Analysis (continued)

7.5.2.5 Modeling of depressions, (Pit Areas and Sand & Gravel Mining) (continued)

In-channel pits
With regard to a hydraulic study involving in-channel pit, an important factor influencing the type of model is the size of the pit. If the pit is very large in comparison to the average channel size, storage and its effect on flow attenuation will be a major consideration. In that case, an unsteady flow analysis considering the volume of water in the hydrograph may be the best choice.

On the other hand, the purpose of the study may govern the kind of model selected. If the purpose is to delineate the floodplain, it might be of interest to compute the most conservative (maximum) water surface elevations. In such a case, a steady-flow model can be used and the water surface computed by ignoring the pit (filling in the pit) in the cross-section. The filled-in area should have a Manning's n similar to the areas upstream and downstream of the pit.

Off-channel pits
For off-channel pits, the major consideration is whether the stream flow will be diverted into the pit. This is affected by the presence of levees, the size of the pit, and the magnitude and duration of the flood. If levees separate the main channel for the pits and will hold during the flood under consideration, the area beyond the levees can be ignored or blocked off. If the levees would fail, or if the flood would overtop the levees, two situations need to be considered.

In one situation, the pit area will be storing water but not actively conveying flow downstream. In this case the pit should be considered as an ineffective flow area. In the second situation, the pit area is actively conveying flow and will have to be modeled as a part of the overbank flow path.

7.5.3 Single Section Analysis

The single-section analysis method (slope-area method) is simply a solution of Manning's Equation for the normal depth of flow given the discharge and cross-section properties including geometry, slope, and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either artificial or stream channels. Nevertheless, the single-section method is often used to design artificial channels for uniform flow as a first approximation, and to develop a stage-discharge rating curve in a stream channel for tailwater determination at a culvert or storm drain outlet.

A stage-discharge curve is a graphical relationship of streamflow depth or elevation to discharge at a specific point on a stream. The stage-discharge curve can be determined as follows:

- Select the typical cross-section at or near the location where the stage-discharge curve is needed.

- Subdivide cross-section and assign n-values to subsections as described in Section 7.5.2.1

7.5 Hydraulic Analysis (continued)

7.5.3 Single Section Analysis (continued)

- Estimate water-surface slope. Since uniform flow is assumed, the average slope of the streambed can usually be used.

- Apply a range of incremental water surface elevations to the cross-section.

- Calculate the discharge using Manning's equation for each incremental elevation. A graphical technique such as that given in Figure 7-12 or a nomograph as in Figure 7-13 can be used for trapezoidal and prismatic channels. For non-prismatic channels, the channel is subdivided into sections; the total discharge at each elevation is the sum of the discharges from each subsection at that elevation. In determining hydraulic radius, the wetted perimeter should be measured only along the solid boundary of the cross-section and not along the vertical water interface between subsections.

- After the discharge has been calculated at several incremental elevations, a plot of stage versus discharge can be made. This plot is the stage-discharge curve and it can be used to determine the water-surface elevation corresponding to the design discharge or other discharge of interest.

An example application of the stage-discharge curve procedure is presented in Appendix B.

In stream channels the transverse variation of velocity in any cross-section is a function of subsection geometry and roughness and may vary considerably from one stage and discharge to another. It is important to know this variation for purposes of designing erosion control measures and locating relief openings in highway fills, for example. The best method of establishing transverse velocity variations is by current meter measurements. If this is not possible, the single-section method can be used by dividing the cross section into subsections of relatively uniform roughness and geometry. It is assumed that the energy grade line slope is the same across the cross section so that the total conveyance K_T of the cross section is the sum of the subsection conveyances. The total discharge is then $K_T S^{1/2}$ and the discharge in each subsection is proportional to its conveyance. The velocity in each subsection is obtained from the continuity equation, $V = Q/A$.

Alluvial channels present a more difficult problem in establishing stage-discharge relations by the single-section method because the bed itself is deformable and may generate bed forms such as ripples and dunes in lower regime flows. These bed forms are highly variable with the addition of form resistance, and selection of a value of Manning's n is not straightforward. Instead, several methods outlined in (Vanoni, 1977) have been developed for this case (Einstein-Barbarossa; Kennedy-Alam-Lovera; and Engelund) and should be followed unless it is possible to obtain a measured stage-discharge relation.

7.5 Hydraulic Analysis (continued)

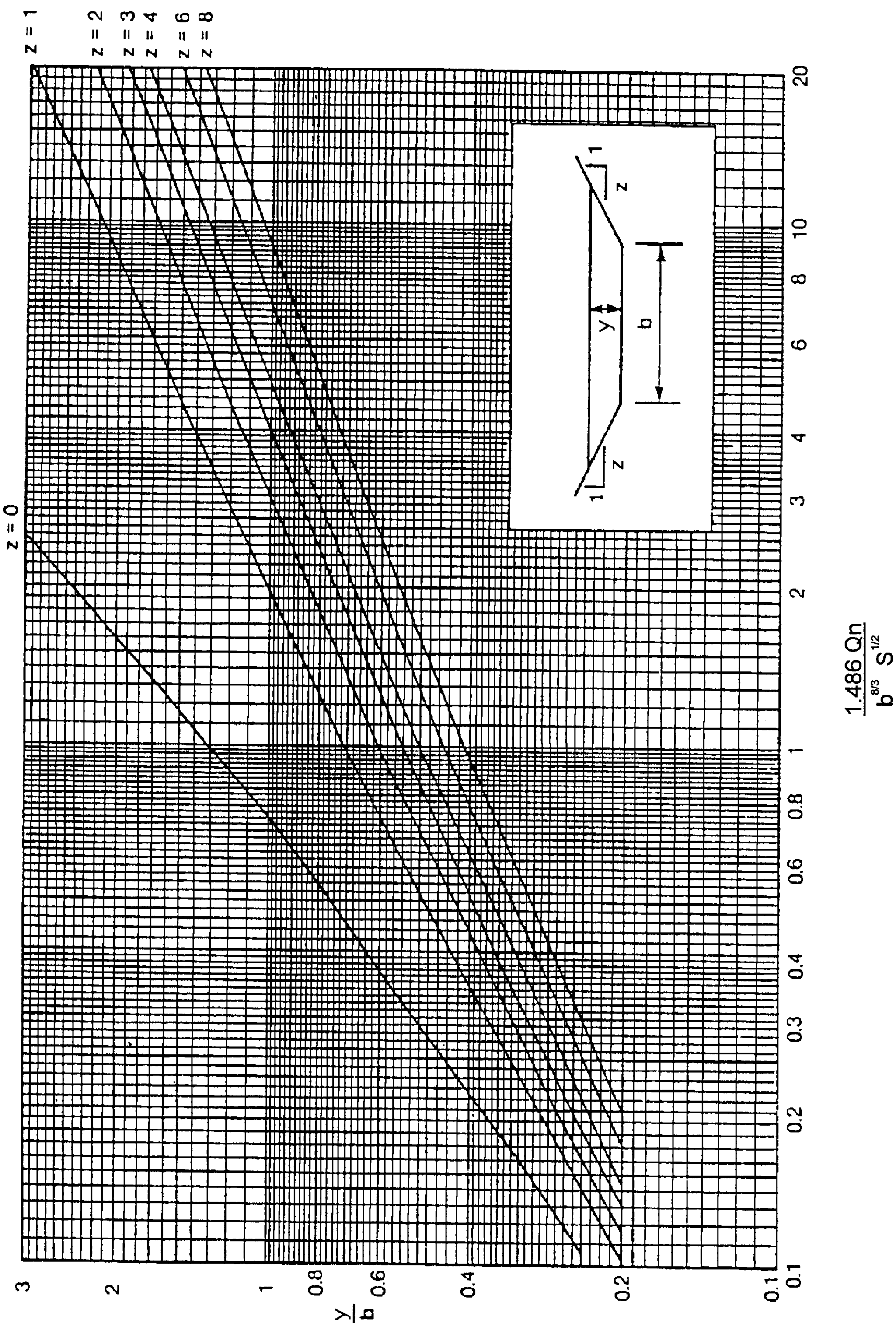


Figure 7-12 Trapezoidal Channel Capacity Chart

7.5 Hydraulic Analysis (continued)

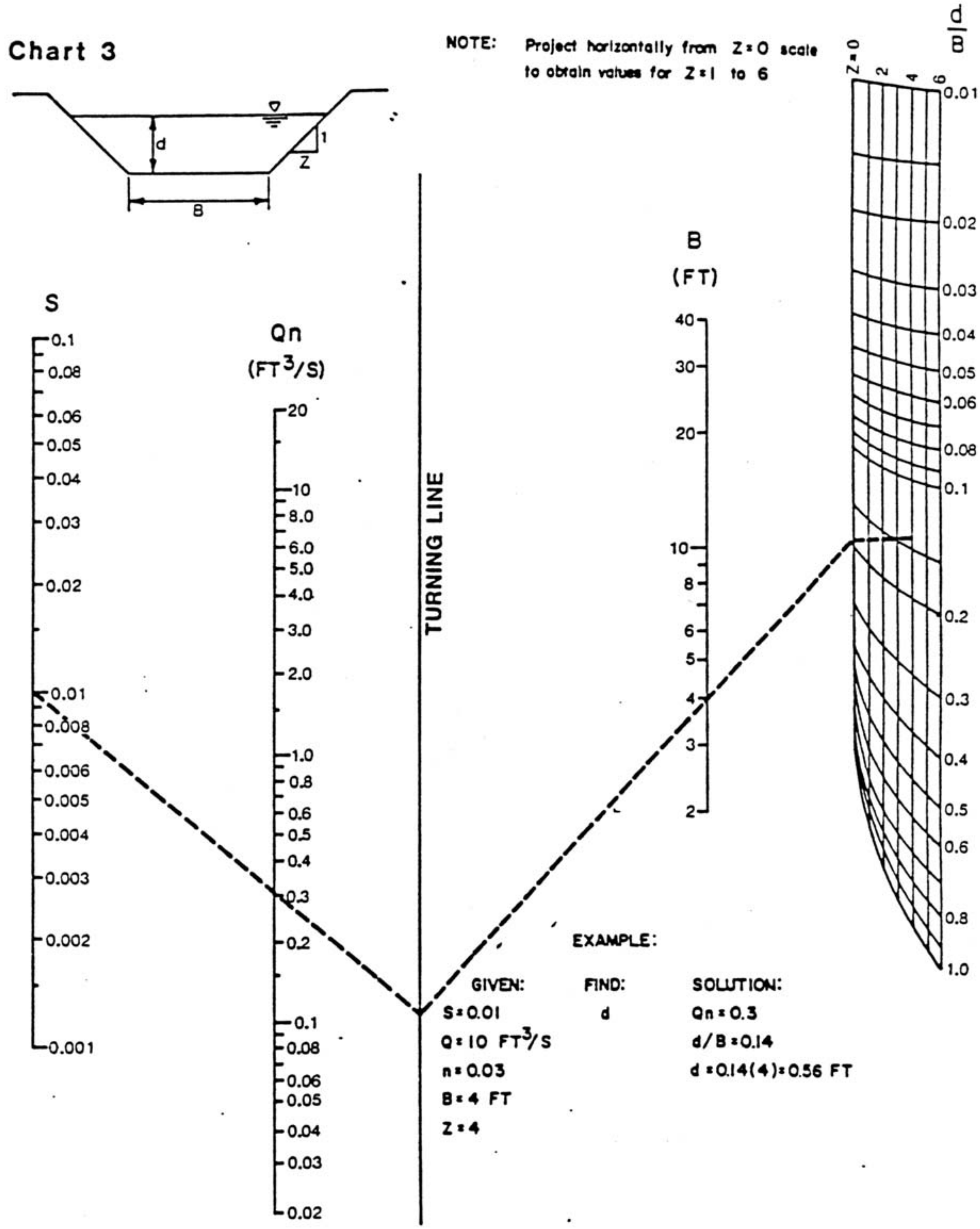


Figure 7-13 Nomograph For Normal Depth
Source: HEC-15

7.5 Hydraulic Analysis (continued)

7.5.4 Step-Backwater Analysis

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned, and for analyzing how far upstream the water surface elevations are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the US Army Corps of Engineers (USACOE) HEC-RAS be used.

7.5.4.1. Step-Backwater Models

The HEC-RAS program developed by the USA Corps of Engineers is widely used for calculating water surface profiles for steady gradually varied flow in a natural or man-made channel. This program is for fixed bed, one-dimensional flow. Both subcritical and supercritical flow profiles can be calculated. The effects of bridges, culverts, weirs, and structures in the floodplain may be also considered in the computations. This program is also designed for application in flood plain management and flood insurance studies.

7.5.4.2 Step-Backwater Methodology

The computation of water surface profiles by HEC-RAS is based on the standard step method in which the stream reach of interest is divided into a number of subreaches by cross-sections spaced such that the flow is gradually varied in each subreach. The energy equation is then solved in a step-wise fashion for the stage at one cross-section based on the stage at the previous cross-section.

The method requires definition of the geometry and roughness of each cross section as discussed in Section 7.5.1. Manning's n values can vary both horizontally across the section as well as vertically. Expansion and contraction head loss coefficients, variable main channel and overbank flow lengths, and the method of averaging the slope of the energy grade line can all be specified.

To amplify on the methodology, the energy equation is repeated from Section 7.4.4:

$$\mathbf{h_1 + \acute{a}_1(V_1^2/2g) = h_2 + \acute{a}_2(V_2^2/2g) + h_L} \qquad \textbf{(7.5.1)}$$

Where:

h_1 and h_2 are the upstream and downstream stages, respectively, ft
$\acute{a}$ = kinetic energy correction coefficient
V = mean velocity, ft/s
h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance, ft

7.5 Hydraulic Analysis (continued)

7.5.4.2 Step-Backwater Methodology (continued)

The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow y, i.e., h = z+y. The energy equation is solved between successive stream reaches with nearly uniform roughness, slope, and cross-sectional properties.

The total head loss is calculated from:

$$\mathbf{h_L = K_m[(\acute{a}_1V_1^2/2g) - (\acute{a}_2V_2^2/2g)] + S_eL} \qquad \mathbf{(7.5.2)}$$

Where:

K_m = the minor loss coefficient
S_e = the mean slope of the energy grade line evaluated from Manning's equation and a selected averaging technique (Shearman, 1990 and HEC-RAS), ft/ft

These equations are solved numerically in a step-by-step procedure called the Standard Step Method from one cross-section to the next.

The default values of the minor loss coefficient K_m are 0.1 for contractions and 0.3 for expansions in HEC-RAS.

7.5.4.3 Profile Computation

Water surface profile computation requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section, but in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can either be found by the single-section method (slope-area method) or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the several profiles do not converge, then the stream reach may need to be extended downstream, or a shorter cross-section interval should be used, or the range of starting water-surface elevations should be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis (see Figure 7-14). Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the desired locations, and continued a sufficient distance upstream to accurately determine the impact of the conditions under considerations on upstream water surface profiles (see Figure 7-15).

7.5 Hydraulic Analysis (continued)

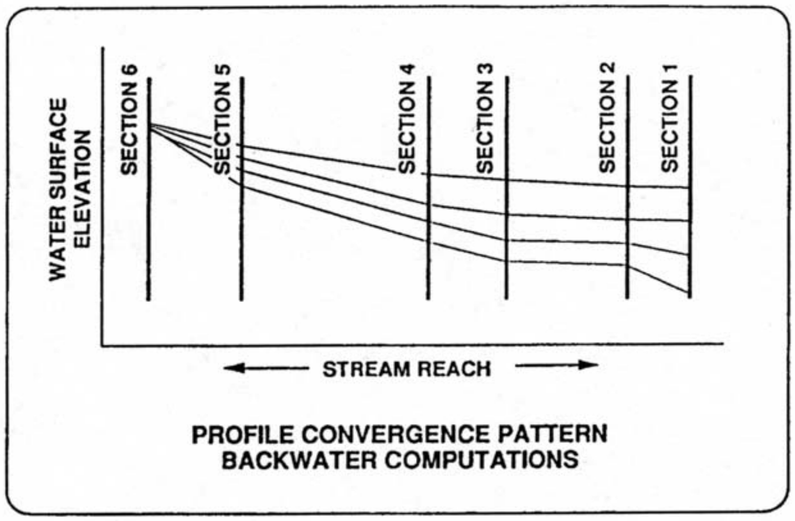


Figure 7-14 Profile Convergence Pattern Backwater Computation

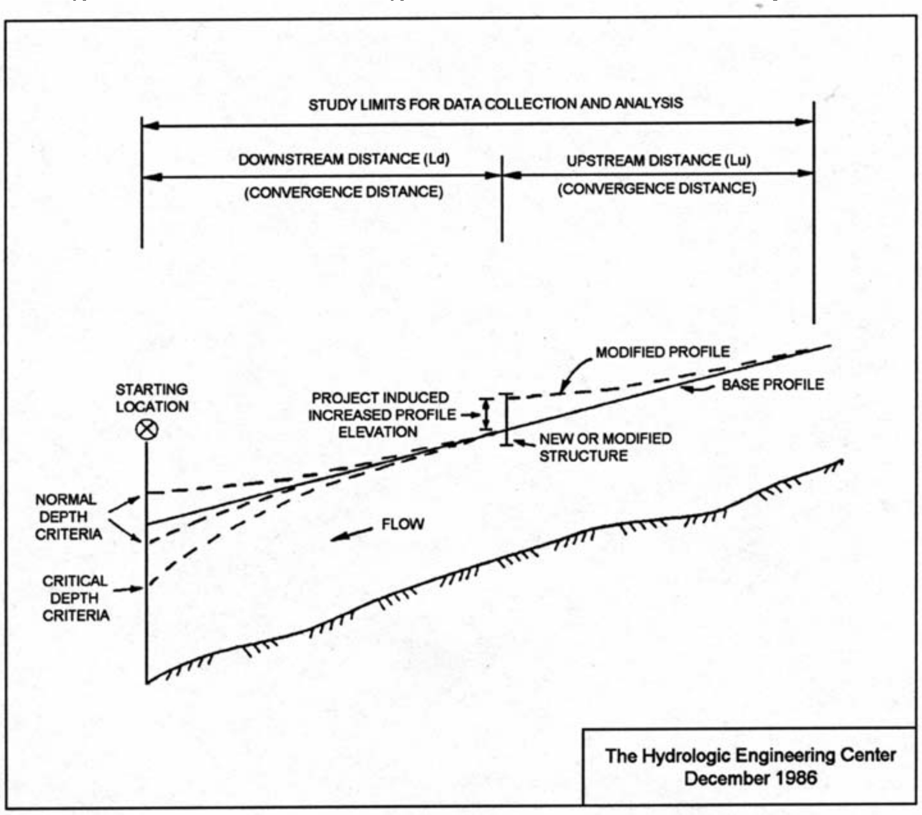


Figure 7-15 Profile Study Limits

Source: USACOE, 1986

7.5 Hydraulic Analysis (continued)

7.5.4.3 Profile Computation (continued)

The USA Corps of Engineers (USACOE, 1986) developed equations for determining upstream and downstream reach lengths as follows:

$$\mathbf{Ldn = 8{,}000\ (HD^{0.8}/S)} \qquad \mathbf{(7.5.3)}$$

$$\mathbf{Lu = 10{,}000\ [(HD^{0.6})(HL^{0.5})]/S} \qquad \mathbf{(7.5.4)}$$

Where:

Ldn = downstream study length (along main channel), ft (for normal depth starting conditions)

Lu = estimated upstream study length (along main channel), ft (required for convergence of the modified profile to within 0.1 feet of the base profile)

HD = average hydraulic depth (1-percent chance event flow area divided by the top width), ft

S = average reach slope, ft/mile

HL = headloss ranging between 0.5 and 5.0 feet at the channel crossing structure for the 1-percent chance flood, ft

References (Davidian, 1984 and USACOE, 1986) are very valuable sources of additional guidance on the practical application of the step-backwater method to highway drainage problems involving open-channels. These references contain more specific guidance on cross-section determination, location, and spacing and stream reach determination. Reference (USACOE, 1986) investigates the accuracy and reliability of water surface profiles related to n-value determination and the survey or mapping technology used to determine the cross-section coordinate geometry.

7.5.5 Modeling and Review Guidance

The following suggestions are to be considered in developing and reviewing HEC-RAS water surface profile models.

Review the warning messages, notes and cautions. Identify any need for additional cross sections. Check the graphical and tabular output. Look for unexpected results in water surface profile, velocity, flow distribution, top width, divided flow and length between cross sections including main channel and overbanks.

The cross-section spacing should be determined based on the change in slope of the ground profile of the watercourse as well as the change in width of the watercourse. In general the cross section spacing is determined by the need to adequately account for the energy losses (friction, flow expansion, and flow contraction) between consecutive cross sections.

7.5 Hydraulic Analysis (continued)

7.5.5 Modeling and Review Guidance (continued)

A review of the energy slope should be undertaken to see if rapid changes occur. If the slope increases or decreases rapidly between consecutive cross sections it usually indicates the need for decreasing the cross-section spacing by adding additional cross sections.

The use of critical depth at an isolated cross-section may indicate an error in geometry of the cross section. If no coding error is found, this may be due to a large change in energy slope. This may indicate a need for additional cross sections or that ineffective flow areas need to be defined. Several cross sections with critical flow may be an indication of supercritical flow. This may require a mixed flow run in HEC-RAS. If this is a FEMA model, this result may be computationally adequate, as FEMA does not permit the use of supercritical profiles in alluvial channels.

A drastic change in top width may require a check of flow paths. Again, additional cross sections may be necessary. The sudden change in top width may also indicate an error in the coding of levees, not specifying areas of ineffective flow, or blocked obstructions.

A large change in the distribution of flow in the channel and overbanks may indicate a need for additional sections.

Indications of divided flow should be checked for consistency with the topography. The divided flow should be hydraulically connected to be run in a single model.

Skewed cross sections should be corrected by using the projected length. The projected length is found by projecting the skewed length onto a plane perpendicular to the direction of flow. In some cross sections the skew may apply only to a part of the cross section such as the channel, and not the overbanks.

The output should be reviewed for warnings about vertical extensions of the ends of the cross section. These warnings occur when the computed water surface elevation exceeds the ends of the cross section. The cross section should be extended based on topographical data as appropriate.

Multiple profile runs should be checked to ensure that there are no conflicts in the modeling requirements for the various profiles. For example, the definition of ineffective flow areas and roughness values for modeling a low flow situation may not be applicable for a high flow situation.

7.6 Design Procedure

7.6.1 General

The design procedure for channels involves two parts. The first part involves the computation of the channel section that carries the design discharge. The second part involves evaluating the degree of protection required for a desirable maintenance and stability performance. The capacity analysis is performed using the principles of flow and Manning's equation. The stability is determined by comparison of the predicted velocity with the permissible velocity for the type of channel lining to be used.

The design procedure for all types of channels has some common elements as well as some substantial differences. This section will outline a process for assessing a natural stream channel and a more specific design procedure for roadside channels.

7.6.2 Stream Channels

The analysis of a stream channel in most cases is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the highway in such a manner that will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining flood plain (see Section 7.2).

Although the following step-by-step procedure may not be appropriate for all possible applications, it does outline a process that will usually apply.

Step 1 Assemble site data and project file.

A. Data Collection.
- Topographic, site, and location maps
- Roadway profile
- Photographs
- Field reviews
- Design data at nearby structures
- Gaging records

B. Studies by other agencies.
- Flood insurance studies
- Floodplain studies
- Watershed studies

7.6 Design Procedure (continued)

7.6.2 Stream Channel (continued)

C. Environmental constraints.
- Floodplain encroachment
- Floodway designation
- Habitat
- Commitments in review documents

D. Design criteria.
- See Section 7.3.

Step 2 Determine the project scope.

A. Determine level of assessment.
- Stability of existing channel
- Potential for damage
- Sensitivity of the stream

B. Determine type of hydraulic analysis.
- Qualitative assessment
- Single-section analysis
- Step-backwater analysis

C. Determine additional survey information.
- Extent of streambed profiles
- Locations of cross sections
- Elevations of flood-prone property
- Details of existing structures
- Properties of bed and bank materials

Step 3 Evaluate hydrologic variables.

A. Compute discharges for selected frequencies.

Step 4 Perform hydraulic analysis.

A. Single-section analysis (7.5.3).
- Select representative cross section (7.5.2)
- Select appropriate n values (Table 7-1)
- Compute stage-discharge relationship

7.6 Design Procedure (continued)

7.6.2 Stream Channel (continued)

Step 4 Perform hydraulic analysis. (continued)

B. Step-backwater analysis (7.5.4).

C. Calibrate with known high water.

Step 5 Perform stability analysis.

A. Geomorphic factors. (long-term degradation, low-flow incisement)

B. Hydraulic factors. (bed forms, bend scour)

C. Stream response to change.

Step 6 Design countermeasures.

A. Criteria for selection.
- Erosion mechanism
- Stream characteristics
- Construction and maintenance requirements
- Vandalism considerations
- Cost

B. Types of countermeasures.
- Meander migration countermeasures
- Bank stabilization (Bank Protection Chapter)
- Bend control countermeasures
- Channel braiding countermeasures
- Degradation countermeasures
- Aggradation countermeasures

C. For additional information.
- HEC-20 Stream Stability
- Highways in the River Environment
- See Reference List

Step 7 Documentation.
- Prepare report and file with background information.

7.6 Design Procedure (continued)

7.6.3 Roadside Channels

A roadside channel is defined as an open channel usually paralleling the highway embankment and within the limits of the highway right-of-way. It is normally trapezoidal or V-shaped in cross section and often lined with grass or a special protective lining.

The primary function of roadside channels is to collect surface runoff from the highway and areas that drain to the right-of-way and to convey the accumulated runoff to an acceptable outlet point.

A secondary function of a roadside channel may be to drain subsurface water from the base of the roadway to prevent saturation and loss of support for the pavement or to provide a positive outlet for subsurface drainage systems such as pipe underdrains.

The alignment, cross section, and grade of roadside channels are usually constrained to a large extent by the geometric and safety standards applicable to the project. These channels should accommodate the design runoff in a manner that considers the safety of motorists, minimizes future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects.

7.6.3.1 Step-By-Step Procedure

Each project is unique, but the following six basic design steps are normally applicable:
In order to obtain the optimum roadside channel system design, it may be necessary to make several trials of the procedure before a final design is achieved.

Step 1 Establish a roadside plan.

A. Collect available site data.

B. Obtain or prepare existing and proposed plan-profile layout including highway, culverts, bridges, and other elements which affect the design.

C. Determine and plot on the plan the locations of natural basin divides and roadside channel outlets. An example of a roadside channel plan/profile is shown in Figure 7.16.

D. Perform the layout of the proposed roadside channels to minimize diversion flow lengths.

7.6 Design Procedure (continued)

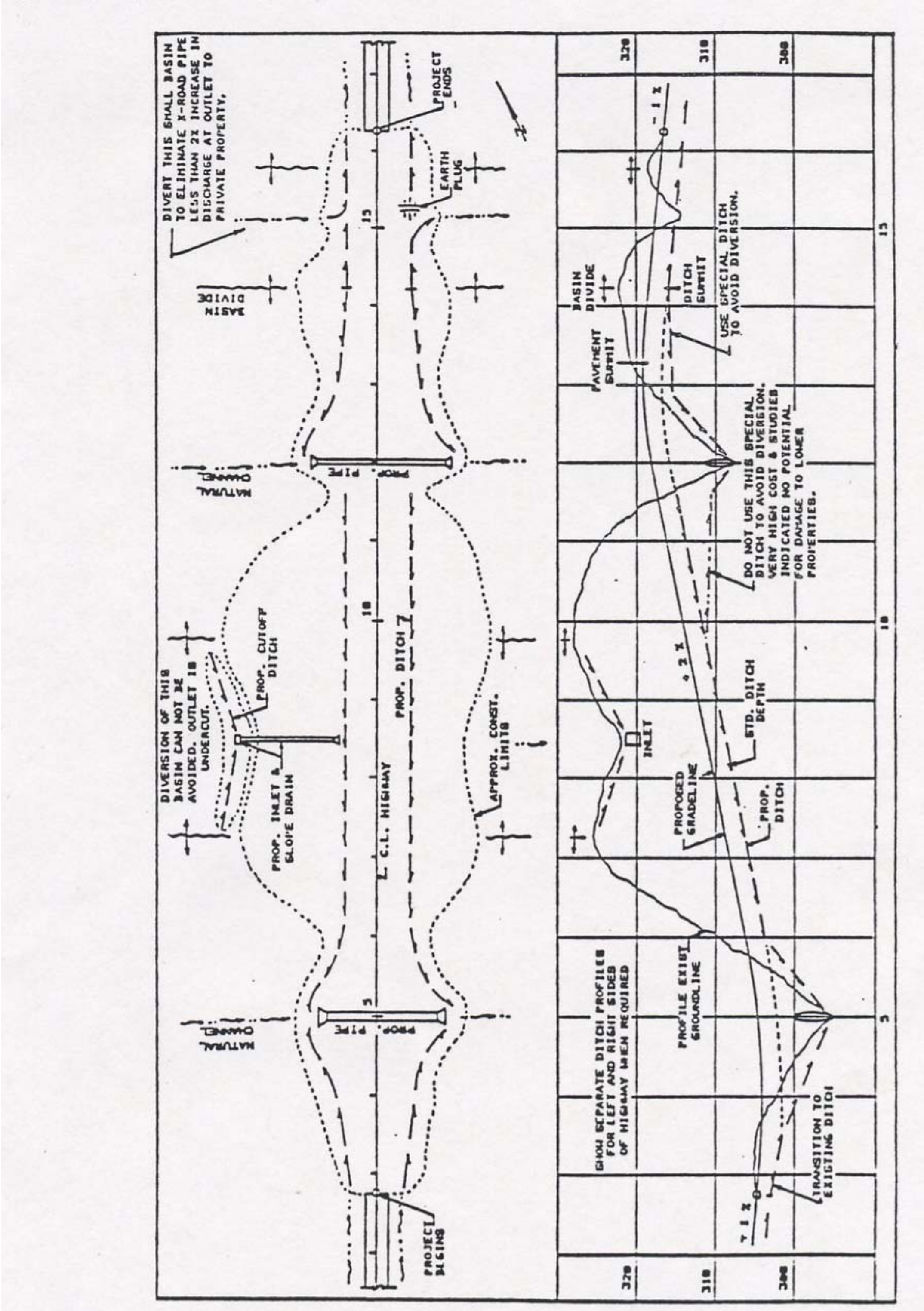


Figure 7-16 Sample Roadside Channel

7.6 Design Procedure (continued)

7.6.3.1 Step-By-Step Procedure (continued)

Step 2 Obtain or establish cross section data.

A. Provide channel depth adequate to drain the subbase and minimize freeze-thaw effects.

B. Choose channel side slopes based on geometric design criteria including safety, economics, soil, aesthetics, and access.

C. Establish bottom width of trapezoidal channel.

D. Identify features that may restrict cross section design:
- right-of-way limits,
- trees or environmentally sensitive areas,
- utilities, and
- existing drainage facilities.

Step 3 Determine initial channel grades.

A. Plot initial grades on plan-profile layout. (Slopes in roadside ditches in cut are usually controlled by highway grades.)

B. Provide minimum grade of 0.3% to minimize ponding and sediment accumulation.

C. Consider influence of type of lining on grade.

D. Where possible, avoid features that may influence or restrict grade, such as utility locations.

Step 4 Check flow capacities and adjust as necessary.

A. Compute the design discharge at the downstream end of a channel segment.

B. Set preliminary values of channel size, roughness coefficient, and slope.

C. Determine maximum allowable depth of channel including freeboard.

7.6 Design Procedure (continued)

7.6.3.1 Step-By-Step Procedure (continued)

Step 4 (continued)

D. Check flow capacity using Manning's Equation and single-section analysis.

E. If capacity is inadequate, possible adjustments are as follows:
- increase bottom width,
- make channel side slopes flatter,
- make channel slope steeper,
- provide smoother channel lining, and
- install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity.

F. Provide smooth transitions at changes in channel cross sections.

G. Provide extra channel storage where needed to replace floodplain storage and/or to reduce peak discharge.

Step 5 Determine channel lining/protection needed (HEC-15).

More details on channel lining design may be found in HEC-15 including consideration of channel bends, steep slopes, and composite linings.

A. Select a lining and determine the permissible shear stress $\hat{o}_p$ in lbs/ft^2 from Table 7C-1 and/or Table 7D-1.

B. Estimate the flow depth and choose an initial Manning's n value from Table 7A-1 or from Table 7B-1.

C. Calculate normal flow depth y_o (ft) at design discharge using Manning's Equation and compare with the estimated depth. If they do not agree, repeat steps 5B and 5C.

D. Compute maximum shear stress at normal depth as:

$$\hat{o}_d\ (lbs/ft^2) = 62.4\ y_o\ S$$

7.6 Design Procedure (continued)

7.6.3.1 Step-By-Step Procedure (continued)

Step 5 (continued)

E. If $\hat{o}_d < \hat{o}_p$ then lining is acceptable. Otherwise consider the following options:

- decrease slope in combination with drop structures if necessary,
- increase channel width and/or flatten side slopes.
- choose a more resistant lining, such as concrete, gabions, or other more rigid lining either as full lining or composite,

Step 6 Analyze outlet points and downstream effects.

A. Identify any adverse impacts to downstream properties that may result from one of the following at the channel outlet:

- increase or decrease in discharge,
- increase in velocity of flow,
- confinement of sheet flow,
- change in outlet water quality, or
- diversion of flow from another watershed.

B. Mitigate any adverse impacts identified in 6A. Possibilities include:

- enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel,
- install velocity control structures,
- increase capacity and/or improve lining of downstream channel,
- install sophisticated weirs or other outlet devices to redistribute concentrated channel flow, and
- install sedimentation/infiltration basins,
- eliminate diversions which result in downstream damage and which cannot be mitigated in a less expensive fashion.

7.6.3.2 Channel Layout Considerations

- A minimum bottom width with access for maintenance equipment of 8 feet may be necessary.

- Turnaround points may be required.

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Appendix 7-A Manning's n, Natural Channels

Table 7A-1

Manning's Roughness Coefficient, n
UNIFORM FLOW

UNLINED CHANNLES

Type Of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform	0.016	0.018	0.020
1. Clean, recently completed	0.018	0.022	0.025
2. Clean, after weathering	0.022	0.025	0.030
3. Gravel, uniform section, clean	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense Weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Appendix 7-A Manning's n, Natural Channels

Table 7A-1 (continued)

Manning's Roughness Coefficient, n
UNIFORM FLOW

Type Of Channel and Description	Minimum	Normal	Maximum
NATURAL STREAMS			
1. Minor streams (top width at flood stage < 100 ft)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and some stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stands of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
2. Flood Plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160

Appendix 7-A Manning's n, Natural Channels

Table 7A-1 (continued)

Manning's Roughness Coefficient, n
UNIFORM FLOW

Type Of Channel and Description	Minimum	Normal	Maximum
NATURAL STREAMS			
2. Flood Plains			
d. Trees			
1. Dense Willows, summer, straight	1.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of spouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flooded stage reaching branches	0.100	0.120	0.160

Type Of Channel and Description	Minimum	Normal	Maximum
3. Major Streams (top width at flood stage > 100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	.....	0.060
b. Irregular and rough section	0.035	.....	0.100

Appendix 7-B Manning's n, Channel Linings

Table 7B-1
Manning's Roughness Coefficient, n (Uniform Flow)

Lining Category	Lining Type	Depth Ranges 0 - 0.5 ft.	0.5-2.0 ft.	>2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	-	0.078	0.040
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
	Wire-tied			
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.

*Some "temporary" linings become permanent when buried.

Appendix 7-C Permissible Velocities

Table 7C-1 Permissible Velocities

Grass and Earth-Lined Channels

Channel Slope	Lining	Permissible Velocity*
0-5%	Bermuda grass	6 ft/sec
	Reed canary grass	5 ft/sec
	Tall fescue	5 ft/sec
	Kentucky bluegrass	5 ft/sec
	Grass-legume mixture	4 ft/sec
	Red fescue	4 ft/sec
	Redtop	4 ft/sec
	Sericea lespedeza	4 ft/sec
	Annual lespedeza	4 ft/sec

Channel Slope	Lining	Permissible Velocity*
	Small grains (temp)	
5-10%	Bermuda grass	5 ft/sec
	Reed canary grass	4 ft/sec
	Tall fescue	4 ft/sec
	Kentucky bluegrass	4 ft/sec
	Grass-legume mixture	3 ft/sec
>10%	Bermuda grass	4 ft/sec
	Reed canary grass	3 ft/sec
	Tall fescue	3 ft/sec
	Kentucky bluegrass	3 ft/sec

- For highly erodible soils, decrease permissible velocities by 25%

Bare Soil

Permissible Velocities for Water Flow conditions

SOIL Types	Clear Water	w/ Fine Silts	w/ Sand & Gravel
Fine Sand (noncolloidal)	1.5 ft/sec	2.5 ft/sec	1.5 ft/sec
Sandy Loam (noncolloidal)	1.7 ft/sec	2.5 ft/sec	2.0 ft/sec
Silt Loam (noncolloidal)	2.0 ft/sec	3.0 ft/sec	2.0 ft/sec
Ordinary Firm Loam	2.5 ft/sec	3.5 ft/sec	2.2 ft/sec
Fine Gravel	2.5 ft/sec	5.0 ft/sec	3.7 ft/sec
Graded, Loam to Cobbles	3.7 ft/sec	5.0 ft/sec	5.0 ft/sec
Graded, Silt to Cobbles (noncolloidal)	4.0 ft/sec	5.5 ft/sec	5.0 ft/sec
Alluvial Silts (noncolloidal)	2.0 ft/sec	3.5 ft/sec	2.0 ft/sec
Alluvial Silts (colloidal)	3.7 ft/sec	5.0 ft/sec	3.0 ft/sec
Coarse Gravels (noncolloidal)	4.0 ft/sec	6.0 ft/sec	6.5 ft/sec
Cobbles and Shingles	4.0 ft/sec	6.0 ft/sec	6.5 ft/sec
Shales and Hard Pan	6.0 ft/sec	6.0 ft/sec	5.0 ft/sec

Source: Special Committee on Irrigation Research, American Society of American Civil Engineers, 1926

Appendix 7-D Shear Stress Capacity

Table 7D-1

Summary Of Shear Stress For Various Protection Measures

Protective Cover	Underlying Soil	t_C (lb/ft$_2$)
Gravel	D_{50} = 1 in	0.40
	D_{50} = 2 in	0.80
Rock	D_{50} = 6 in	2.50
	D_{50} = 12 in	5.00
6 in Gabions	Type I	35
4 in Geoweb	Type I	10
Soil Cement (8% cement)	Type I	>45
Concrete construction Blocks, granular filter underlayer	Type I	>20
Wedge-shaped blocks with drainage slot	Type I	>25

Source: FHWA-RD-89-110, HEC-15

Appendix 7-E Water Surface Profile Computation

Example Problem

A sample computation is taken from "Hydrologic Engineering Methods For Water Resources Development - Volume 6, Water Surface Profiles", The Hydrologic Engineering Center, Corps of Engineers, U.S. Army, Davis, California.

A convenient form for use in calculating water surface profiles is shown in Figure 7-E-1. In summary, columns 2 and 4 through 12 are devoted to solving Manning's Equation to obtain the energy loss due to friction, columns 13 and 14 contain calculations for the velocity distribution across the section, columns 15 through 17 contain the average kinetic energy, column 18 contains calculations for "other losses" (expansion and contraction losses due to interchanges between kinetic and potential energies as the water flows), and column 19 contains the computed change in water surface elevation. Conservation of energy is accounted for by proceeding from section to section down the computation form.

Column 1 - CROSS SECTION NO., is the cross-section identification number. Miles upstream from the mouth are recommended.

Column 2 - ASSUMED, is the assumed water surface elevation which must agree with the resulting computed water surface elevation within ± .05 feet, or some allowable tolerance, for trial calculations to be successful.

Column 3 - COMPUTED, is the rating curve value for the first section, but thereafter, is the value calculated by adding WS to the computed water surface elevation for the previous cross section.

Column 4 - A, is the cross section area. If the section is complex and has been subdivided into several parts (e.g., left overbank, channel and right overbank) use one line of the form for each sub-section and sum to get A_t, the total area of cross section.

Column 5 - R, is the hydraulic radius. Use the same procedure as for column 4 if section is complex, but do not sum subsection values.

Column 6 - $R_{2/3}$, is 2/3 power of hydraulic radius.

Column 7 - n, is Manning coefficient of channel roughness.

Column 8 - K, is conveyance and is defined as ($C_m A R_{2/3}/n$) where C_m is 1.49 for English units. If the cross section is complex, sum subsection K values to get K_t.

Column 9 - K_t, is average conveyance for the reach, and is calculated by $0.5(K_{td} + K_{tu})$ where subscripts D and U refer to downstream and upstream ends of the reach, respectively.

Appendix 7-E Water Surface Profile Computation

Column 10 - S_f, is the average friction slope through the reach determined by $(Q/K_t)^2$.

Column 11 - L, is the distance between cross sections: different values may be used in each strip.

Column 12 - h_f, is energy loss due to friction through the reach and is calculated by $h_f = (Q/K)^2L$.

Column 13 - $K(K/A)^2$, is part of the expression relating distributed flow velocity to an average value. If the section is complex, calculate one of these values for each subsection and sum all subsection values to get a total. If one subsection is used, Column 13 is not needed and (Column 14) equals one.

Column 14 - α, is the velocity distribution coefficient and is calculated by $K(K/A)^2/(K_t/A_t)^2$ where the numerator is the sum of values in Column 13 and the denominator is calculated from K_t and A_t.

Column 15 - V, is the average velocity and is calculated by Q/A_t.

Column 16 - $V^2/2g$, is the average velocity head corrected for flow distribution.

Column 17 - $(\Delta\alpha V^2/2g)$, is the difference between velocity heads at the downstream and upstream sections. A positive value indicates velocity is increasing, therefore, use a contraction coefficient for "other losses". A negative value indicates the expansion coefficient should be used in calculating "other losses".

Column 18 - h_o, is "other losses", and calculated with either C_e or C_c.

Column 19 - WS, is the change in water surface elevation from the previous cross section. It is the algebraic sum of columns 12, 17 and 18.

Appendix 7-E Water Surface Profile Computation

Cross Section No.	Water Surface Elevation		Area	Hydraulic Radius R	$R^{2/3}$	n	K	$\bar{K}$	1000 $\bar{S}_f$	L	h_f	K^3/A^2	α	V	$\alpha V^2/2g$	$\Delta(\alpha V^2/2g)$	h_e	Δ Water Surface Elevation
	Assumed	Computed																
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

Figure 7-E-1 Water Surface Profile Form

Appendix 7-F HEC-RAS Checklist

INPUT CHECKLIST

1. GEOMETRIC DATA

A. Review the Project Limits (limits of data collection). Is enough information (cross-sections) gathered both upstream and downstream of the study reach? For subcritical regime, make sure that the upstream project limit is at a distance where the water surface profile resulting from a channel modification converges with the existing conditions profile (to evaluate any upstream impacts due to project alternatives). The downstream limit should be far enough to prevent any user identified boundary condition from affecting the results within the study reach. For supercritical regime, the roles of the upstream and downstream limits are reversed.

B. Check the river system schematic. Are the various reaches (for a dendritic river system) properly connected? Inspect the location of junctions. Are the flow directions correct? Check the location of flow splits and flow combinations in looped networks (if any).

C. Review the cross section geometry. Does it characterize locations of changes in discharge, slope, shape or roughness, locations where levees begin or end, at bridges, culverts, weirs, or other control structures? Are the cross sections properly oriented (perpendicular to the anticipated flow lines, i.e. approximately perpendicular to the ground contour lines)? Review individual cross-section plots. Does a cross section extend across the entire floodplain? Is each end of the cross section higher than the anticipated maximum water surface elevation? Is the topography of the channel (bank elevation) and floodplain accurately reflected in the geometry of the cross sections?

D. Review the reach lengths (distances between cross sections). Check that the channel reach lengths are correctly determined along the thalweg and the overbank reach lengths are measured along the anticipated path of the center of mass of the overbank flow. Make sure that the cross section properly reflects the stream size, slope, uniformity of cross-section shape, and the purpose of the study.

E. Review the profile plots of channel bed elevations and top of bank elevations for abrupt changes, adverse grade, or other anomalies.

2. FLOW DATA

A. What is the design discharge and how is it derived?

B. Is there existing discharge data (hydrologic record) that may be more appropriate or required for regulatory purposes?

C. Are there any tributaries at which a change in discharge might be expected?

D. Are there multiple discharges (multiple profile runs)? What return interval (event) does each discharge represent?

Appendix 7-F HEC-RAS Checklist

2. FLOW DATA(continued)

E. What is the expect flow regime? Is there a possibility for mixed flow regime?

3. BOUNDARY CONDITIONS

A. What method is used to establish the staring water surface elevation: observed, slope-area, critical, or other? Is this method appropriate based on available information on flow regime and topography (for subcritical flow, boundary conditions are necessary at the downstream project limit; for supercritical flow, boundary conditions are necessary at the upstream project limit.; for mixed flow conditions, boundary conditions are necessary at both project limits.)?

B. If there is not a known starting water surface elevation, prepare a range of user-defined starting elevations to check the sensitivity of the results in the study reach.

4. ENERGY LOSS COEFFICIENTS

A. What are Manning's roughness coefficients are used for the channel and overbank areas? Review available aerial and/or ground photography. Conduct a field reconnaissance. Are the coefficients realistic and representative of vegetation, season change, channel irregularities, channel alignment, channel slope, stage and discharge, and bedforms? Is there a need to model more than three distinct zones within each cross section (left overbank, channel, and right overbank)? Does aerial photography or field review indicate braided channels or other areas with the horizontal variability or roughness? Check if the observed water surface profile information (gaged data and high water marks) is available for the roughness calibration. Compare the adopted Manning's coefficients to those used in other studies for similar stream conditions and/or those obtained from experimental data.

B. What expansion and contraction coefficients are used to evaluate transition losses? Are they representative of the changes in geometry between successive cross sections and flow regime? Do they include energy losses at bridges, culverts, weirs, and other control structures? Make sure that the coefficients applied between two cross sections are specified as part of the data for the upstream cross sections.

5. INEFFECTIVE FLOW AREAS

A. Are non-conveying, flow separation areas where the velocity in the downstream direction is close to zero (e.g. "shadow areas" outside the main flow conveyance zone approaching or exiting a bridge, culvert, or other flow obstacle) modeled as ineffective?

B. Are depressions such as overbank excavations or low grounds where water ponds but is not actively being conveyed, represented as ineffective flow areas?

Appendix 7-F HEC-RAS Checklist

INPUT CHECKLIST (continued)

6. SPECIAL CONDITIONS

Based on review of the input data, note the existence or any indication of the possible existence of the following conditions for further investigation after reviewing the output:

A. Bridges, culverts, weirs, and other control structures
B. Levees
C. Blocked obstructions
D. Distributary or alluvial fan conditions
E. Split and/or divided flow
F. Islands

OUTPUT CHECKLIST

1. KEY HYDRAULIC PARAMETERS

Check the following parameters for consistency are reasonableness:

A. Flow depth

B. Critical flow depth

C. Velocity

D. Velocity Head

E. Area

F. Top width

G. Invert slope

H. Energy slope

These parameters should vary gradually between cross sections. Note any unusual variations and any extreme values that do not seem realistic or are inconsistent with known conditions regarding the stream reach.

2. FLOW CONSISTENCY

A. Check the streamwise variation (from cross section to cross section) in the flow distribution between the channel and the left/right overbank. Does the amount of flow (discharge) in any one area vary from one cross section to the next?

Appendix 7-F HEC-RAS Checklist

OUTPUT CHECKLIST (continued)

2. FLOW CONSISTENCY (continued)

B. Check the later distribution of flow between the channel and the overbanks for each cross section. Does it seem reasonable (e.g. if majority of flow is in one overbank, is this what is expected based on the input review)?

3. ERROR AND WARNING MESSAGES

Review summary of errors, warnings, and notes generated after each run. It is important to note that the user does not have to eliminate all the warning messages. However, it is up to the user to determine whether or not these warnings require additional actions for the analysis.

Some common messages to look for include:

A. If there are consistent warning messages indicating profile defaulting to critical depth, consider modeling the alternative flow regime (subcritical vs. supercritical). Mixed flow regime should be attempted.

B. Each cross section with the energy equation could not be balanced message (so that critical depth was assumed) requires further examinations. This message is often an indication of unstable modeling (due to insufficient number of cross sections and data points, inconsistent flows, inaccuracy of roughness coefficients and energy losses), rather than critical depth flows depths. However, it is up to the user to determine whether critical depth is a legitimate answer which indicates minimum specific energy of the flow (e.g. at a transition from subcritical to supercritical flow, at a sudden constriction in subcritical flow, etc.).

C. If there are any extended cross sections messages (these messages indicate the computed vertical floodplain limits exceed the limits of the cross section), consider obtaining additional ground points from the available topography to “close” the cross section and account for additional flow area. Consider if flow would actually leave the main channel at this location (split flow situation).

D. In case of divided flow messages, it should be first determined whether the water can actually flow on both sides of the dividing land feature at the specified flow rate. If so, this usually requires separate modeling of divided reaches.

E. Messages indicating change in velocity head and/or conveyance ration exceeding allowable limits imply that the flow area are changing abruptly between cross sections and may call for additional cross sections or specification of ineffective flow areas.

F. Any other messages should be examined and either eliminated or justified.

Appendix 7-F HEC-RAS Checklist

OUTPUT CHECKLIST (continued)

4. SPECIAL CONDITIONS

Based on the foregoing review, determine whether the model input or output suggest any of the following special flow conditions:

A. Bridges and/or culverts? Are boundary cross sections properly located? Has the correct computational method been used (energy, momentum, Yarnell, pressure and/or weir)? Are ineffective flow limits properly specified? Check pressure/low flow distributions in the output?

B. Levees? Is flow confined within levees allowing overbank flow only above the levee crest stage?

C. Blocked obstruction? Note that these elements decrease flow area and add wetted perimeter when water comes in contact with them (unlike ineffective flow limits).

D. Distributary or alluvial fan conditions? Does the output indicate consistent occurrence of flows diverging from a common path without rejoining downstream or do flow characteristics indicate a gradually expanding pattern of flow with little or no boundary definition? If so, a distributary type flow pattern may predominate, making one-dimensional modeling impractical or impossible.

E. Split and/or divided flow? Flow overtopping a divide as side weir flow? Have these areas been accounted for using split flow modeling or other approximation to account for lost flow?

F. Islands? Do the model results indicate isolated flood free areas within the floodplain? The occurrence of islands may indicate a flow pattern similar to split or divided flow where the two (or more) separate flow paths around the island must be modeled separately to accurately determine flow profiles.

Appendix 7-G Equilibrium Slopes & Drop Structures

Equilibrium Slopes & Drop Structures

Given a fixed distribution of sediments, the sediment-transport capacity of a stream is dependent on flow velocity and depth. For most streams, transport of all particle sizes of bed material increases, as flow velocity increases, at a rate proportional to the third to fifth power of the velocity. Correspondingly, transport of sediment particles composed of bed material generally decreases as depth increases, while transport increases with decreased depth.

For the purpose of analysis and design, most natural, undisturbed channels can be assumed to be at or near a state of dynamic equilibrium with regard to sediment transport. This means that over a given reach of channel, the sediment-transport capacity of the channel, over the long term, is more or less equal to the sediment supply. The channel bed is therefore stable".

The equilibrium slope for a channel that has an upstream sediment supply that is essential zero can be computed by:

$$S_{eq} = (1.45n/q^{0.11})^2 \tag{7G-1}$$

Where:

S_{eq} = Equilibrium slope with no sediment supply, ft./ft.
n = Manning's roughness coefficient,
q = Channel unit discharge, cfs/ft.

For channels that have some sediment-transport capacity, the equilibrium slope can be computed by

$$S_{eq} = [\{(n_u/n_n)^2 * (Q_{u,10}/Q_{n,10})^{-1.1} * (b_u/b_n)^{0.4} * (1-R_s)^{0.7}] * S_n \tag{7G-2}$$

Where:

S_{eq} = Equilibrium slope
n_u, n_n = Manning's roughness
$Q_{u,10}$, $Q_{n,10}$= Ten year discharge,
b_u, b_n= Channel bottom width, ft
R_s = Reduction factor for sediment supply. This factor is usually assumed to be equal to the ratio of impervious area to the total area of the watershed.
S_n = Natural or existing slope, ft./ft.

The subscripts n or u relate to the natural or urbanized condition.

Appendix 7-G Equilibrium Slopes & Drop Structures

Equilibrium Slopes (continued)

Spacing and depth of grade control structures

If the equilibrium slope of a channel, as determined by either equation above, is flatter than the existing or design slope, a grade-control structure may be needed to limit degradation from exceeding a certain depth at any point along the channel. Grade-control structures are barriers in a channel that prevent the channel bed from degrading at a point located immediately upstream of where they are located. After the channel has reached equilibrium, the bed elevation immediately upstream of the grade control structure is at the design elevation. Downstream of the grade-control structure, the bed is at an “equilibrium” elevation that is lower than the design elevation. For most channels the design process is iterative involving drop height, reach length, and depth of scour downstream of the drop.

Once a drop height is chosen, the reach length, or spacing, between adjacent structures can be computed from

$$L_r = h/(S_{ib}-S_{eq}) \qquad \textbf{(7G-3)}$$

Where:
L_r = Reach length, or spacing, between adjacent grade-control structures, in ft.
H = Drop height, measured at the downstream face, in ft.
S_{ib}= Initial channel bed slope, in ft/ft
S_{eq} = Equilibrium channel bed slope, in ft/ft.

If the initial and equilibrium slopes are approximately the same, the distance between structures will be large. Under such circumstances, grade-control structures may not be necessary.

For economical and technical reasons, grade-control structures should be spaced no closer than twelve times the local scour depth as computed by equation 7G-4 or equation 7G-5 as discussed in Chapter 9, energy dissipators.

$$Z_{lsf} = 1.32(q)^{0.54} (H_t)^{0.225}-TW \qquad \textbf{(7G-4)}$$

$$Z_{lss} = 0.581(q)^{0.667} (h/Y)^{0.411}[1-(h/Y)]^{-0.118} \qquad \textbf{(7G-5)}$$

Grade-control structures may be constructed of un-reinforced concrete walls of no more than two feet high, to very large energy dissipators as discussed in Chapter 11.

Appendix 7-G Equilibrium Slopes & Drop Structures

Equilibrium Slopes (continued)

Example: Spacing and depth of grade-control structures.

A channel is to be constructed in an urbanized area to contain the 100-year discharge. The banks of the channel are to be of shotcrete, the bottom is earth.

Channel characteristics are as follows:

Bottom width = 20 feet
Design slope = 0.006 ft/ft
Side slope 1:1
Manning's n = 0.022

Hydraulic data is

Q_{10} = 350 cfs	Q_{100} = 700 cfs
Y_{10} = 2.1 ft.	Y_{100} = 3.1
V_{10} = 7.7 fps.	V_{100} = 9.7 fps.
Unit discharge, q = 350/20 = 17.5 cfs	Unit discharge, q = 700/20 = 35.0 cfs

As the watershed is considered to be urbanized the equilibrium slope is calculated with equation 6.25.

$S_{eq} = (1.45(n)/q^{0.11}) = (1.45(0.022)/(17.5^{0.11}) = 0.0005$, try a 2-foot drop.

The spacing between drops by equation 7G-3 is

$$L_f = \frac{2.0}{(0.006-0.0005)} = 364 \text{ feet.}$$

Since the drop is 2 feet, and Y_{10} = 2.1 ft. the drop is submerged, the scour is estimated by equation 6.14 as

$$Z_{lss} = 0.581(q_{100})^{0.667}\,(h/Y_{100})^{0.411}[1-(h/Y_{100})]^{-0.118}$$

$$Z_{lss} = 0.581(35)^{0.667}\,(2/3.1)^{0.411}[1-(2/3.1)]^{-0.118}$$

$$Z_{lss} = 5.9 \text{ feet.}$$

Therefore, the total height of the drop structure from the top to the toe should be 5.9 feet plus the two-foot drop height; or 7.9 feet. This dimension does not include any protective depth below the scour.

Appendix 7-G Equilibrium Slopes & Drop Structures

Equilibrium Slopes (continued)

Example: Spacing and depth of grade-control structures. (continued)

If it is desired to keep the depth to less than 6 feet, a one-foot drop results in

$$L_f = \frac{1.0}{(0.006-0.0005)} = 182 \text{ feet.}$$

and

$$Z_{lss} = 0.581(q_{100})^{0.667}\,(h/Y_{100})^{0.411}[1-(h/Y_{100})]^{-0.118}$$

$$Z_{lss} = 0.581(35)^{0.667}\,(1/3.1)^{0.411}[1-(1/3.1)]^{-0.118}$$

Z_{lss} = 4.1 feet. For a total height of 5.1 feet.

This would result in 5-foot high drop structures with one-foot exposed at 180-foot spacing.

Appendix 7-H Concrete Lined Channels

Presented are some general guidelines for concrete channel lining design including minimums where appropriate. The channel lining designer should consult with the appropriate materials engineer regarding lining thicknesses, side slopes, concrete mix design, concrete shrinkage criteria, weephole requirements, and required subgrade treatment.

Thickness:
The channel lining thickness and reinforcement shall be designed for the soil conditions at the project site and should consider any negative pressures that might occur and any collapsing or expansive soils. The minimum thickness for trapezoidal channels should be as shown in the table below.

Minimum Channel Lining Thickness

Mean Water Velocity (ft/sec)	**Slab Thickness (in.) Bottom Slab**	**Side Slope Slab**
Less than 10	**5**	**5**
10 to 15	**6**	**5.5**
15 to 20	**7.5**	**6**
more than 20	**8**	

Note: 6-inch minimum with tied reinforcement or when bottom slab width is 8 or more feet (required to support vehicles).

Joints:
Concrete channel linings should be continuously reinforced Portland cement without expansion or tooled joints. Longitudinal construction joints should be located as required for construction but within the low flow area of the bottom slab. The bottom slab pours should extend a minimum of 1 foot up the side slope. Transverse construction joints should be provided only when concrete placement stops for more than 45 minutes. The reinforcing shall be continuous through the construction joints and through joints with culverts and other hydraulic structures. **Sealed expansion joints with load transfer devices shall be provided at bridge piers, abutments, and other fixed structures.**

Side slopes:
Side slopes on main channels should be designed for soil conditions at the site but should not exceed a desirable slope of 2:1 with an absolute minimum of 1.5:1. If steeper slopes are required, the lining shall be designed as a retaining wall for appropriate lateral earth pressures.

Subgrade treatment should be on a site-specific basis as recommend by the geotechnical engineer. Pressure relief of channel linings should be provided by geotextile or geocomposite drainage strips and 4-inch diameter PVC weepholes through the lining 12 inches above the channel. The spacing of weepholes should be based on subsurface investigation; potential future changes in ground water levels, any structural backfill and any parallel or crossing futilities.

Appendix 7-H Concrete Lined Channels (continued)

Reinforcing:
Reinforcing steel shall be mild steel of either bars or welded wire fabric uniformly distributed. Longitudinal reinforcement should be a minimum of 0.3 percent of the concrete area. The minimum percentage of transverse reinforcement is dependent upon the top width of the channel with 0.20 percent for widths less than 65 feet, 0.25 percent for widths from 65 to 100 feet and 0.30 percent for widths greater than 100 feet.

Clearance:
Reinforcing shall have a minimum of 3-inch clearance to grade and a minimum of 2-inch clearance to exposed surfaces.

Cutoff walls:
Cutoff walls will be required:

- Where new lining abuts an existing concrete or other type of lining which is not continuously reinforced; (a sealed expansion joint should be provided between the new and existing linings)
- At the beginning (upstream end) of a transition section
- At breaks in the channel profile of more than 0.9 percent; and
- At existing structures where the new lining cannot realistically be made continuous with the existing lining.

Cutoff walls are not required to prevent progressive failure in continuously reinforced channels; however, stability of sloped wall lining at transitions where the cross-section shape changes, or at locations where channel slopes change must be evaluated. The change in directions will result in unbalanced force action away from the supporting earth. To prevent local buckling at these locations, cutoff walls rigidly attached to the paving, should be installed to stiffen the lining.

Appendix 7-I Example Computations

Rectangular Channel
1. Normal depth
2. Critical depth

Trapezoidal Channel
1. Normal depth
2. Channel capacity

Circular Channel
1. Normal depth
2. Critical depth
3. Equivalent depth

Appendix 7-I Example Computations (continued)

Rectangular Channel

1. Normal depth

Given: 5 ft. wide channel,
Q= 200 cfs.
n= 0.012
So = 0.03

Find: Normal depth of flow: d_n

For use of chart need following parameter; $Qn/(b^{2.67}\ So^{0.5})$

$Qn/(b^{2.67}\ So^{0.5}) = (200*0.012) / (5^{2.67}\ 0.03^{0.5}) = 2.4 / (73.49*0.1732) = 2.4/12.73 = 0.188$

From channel chart; d/b = 0.365, therefore depth =0.365*5 = 1.83 ft.

2. Critical depth

Find Critical depth:

For a rectangular channel,
$d_c = 0.315((Q/b)^2)^{0.333}$

$d_c = 0.315*((200/5)^2)^{0.333} = 0.315*(1600)^{0.333} = 3.68$ ft.

Appendix 7-I Example Computations (continued)

Trapezoidal Channel

1. Normal depth

Given:

$Q = 800$ cfs
$B = 8$ ft.
$Z = 4$ (Side Slope 4:1)
$n = 0.03$
$S_o = 0.05$

Find normal depth and velocity.

For use of chart need following parameter; $Qn/(b^{2.67}\ So^{0.5})$

$Qn/(b^{2.67}\ So^{0.5}) = (800*0.03) / (8^{2.67}\ 0.05^{0.5}) = 24 / (257.8*0.2236) = 24/57.65 = 0.416$

From channel chart; d/b = 0.34, therefore depth =0.34*8 = 2.72 ft.

For a depth of 2.72, top width = 8.0 + 2.72*2*4 = 29.76 ft.

Area = d(b1 +b2)/2 = 2.72*(8 + 29.76)/2 = 51.35 sq. ft.

Velocity = Q/A = 800/51.35 = 15.6 ft/sec.

2. Channel capacity

Given : $B = 4$ ft.
$d = 2$ ft
$Z = 4$ (Side Slope 4:1)
$n = 0.035$
$S_o = 0.05$

Find velocity and discharge.

$V = (1.486/n)\ R^{0.67}S^{0.5}$

Where R = A/P.
$A= Bd + Zd^2 = 4(2) + 4(2)^2 = 24.0$

$P = b + 2d(Z^2+1)^{0.5} = 4 +2(2)(4^2 +1)^{0.5} = 4 +2(2)(4^2 +1)^{0.5} = 20.49$ ft.

$V = (1.486/0.035)*(1.171)^{0.67}\ (0.05)^{0.5}$

Appendix 7-I Example Computations (continued)

2. Channel capacity (continued)

$R=A/P = 24.0/20.49 = 1.171$

$V = 42.46*(1.112)*(0.2236) = 10.55$

$Q = VA$

$Q = 10.55*24.0 = 253.2$ cfs

Circular channel

1. Normal depth

Given:
Q = 400 cfs
D = 60" = 5.0 ft.
n = 0.012
$S_o = 0.05$

For use of chart 47 need following parameter; $Qn/(D^{2.67}\ S_o^{0.5})$

$Qn/(D^{2.67}\ S_o^{0.5}) = (300*0.012)\ /(5^{2.67}\ 0.05^{0.5}) = 0.22$

From chart d/D = 0.57, $d_n = 0.57*5.0 = 2.85$

2. Critical depth

Given:
Q= 300 cfs.
D = 60" = 5.0 ft.

Using chart critical depth chart, $d_c = 4.7$

3. Equivalent depth

Q= 400 cfs
n = 0.012
$S_o = 0.05$

Appendix 7-I Example Computations (continued)

3. Equivalent depth (continued)

Find equivalent depth, d_e

$d_e = (A/2)^{0.5}$

$$d/D = \frac{\text{depth of Flow}}{\text{Diameter of Pipe}} = \frac{2.85}{5.0} = 0.57$$

From d/d vs A, $A = 0.4625D^2$

A = 0.4625*(25) = 11.56 sq.ft.

$d_e = (11.56/2)^{0.5}$

$d_e = 2.40$ ft.